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OF CIVIL ENGINEERS

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TOLERANCE OF PLANTS TO SALINITY

By Leon Bernstein[1]

SYNOPSIS

Soluble salts tend to accumulate in soils in arid regions and are a potential hazard wherever irrigation is practiced. Plant response is primarily determined by the osmotic pressure of the soil solution to which the electrical conductivity of the saturation extract of soils is closely enough related to provide a useful index of salinity. Specific ion effects, involving nutrition or direct toxicity, affect certain crops under some conditions. Climate, irrigation frequency, and other management practices may modify the salt tolerance of plants. Varietal differences in salt tolerances are not usually of practical importance for most crops, but rootstock specificities regulating the accumulation of potentially toxic ions are of major importance for many fruit crops. Because soil management factors profoundly influence the degree of salt accumulation, plant response to a given irrigation water cannot be predicted without taking into account the probable effects of the conditions under which the water will be used on the resultant level of soil salinity. Soil solutions are often five or ten times more concentrated than the applied irrigation water. The use of more saline water requires that larger quantities of water be applied to promote leaching and to minimize the relative degree of salt accumulation in the root zone of the crop.

Plants require certain mineral elements for normal growth and absorb them mostly as soluble salt constituents (ions) from the soil; however, ex-

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[1] Plant physiologist, U. S. Salinity Lab., Soil and Water Conservation Research Div., Agric. Research Service, U. D. Dept. of Agric., Riverside, Calif.

cessive salt concentrations are harmful. Soluble salts are continually being produced in the soil by the weathering of rock and by other processes but are carried away by percolating and flowing water. In humid regions, leaching of salts by rainfall is a major factor determining the need for periodic liming and fertilization to replace leached mineral elements. In arid regions, however, rainfall is slight, and soluble salts tend to accumulate. Irrigation waters, unlike rain, may also bring appreciable quantities of soluble salts into the soil. Special salinity problems sometimes develop along seacoasts because of inundations by seawater, tidal action, or salt-water intrusions. Excessive fertilization in greenhouses or intensively farmed areas may also cause salinity problems. Thus, although soluble salts as a class of substances are essential for plant growth, a number of factors, chief among which are

FIG. 1.—SALT-AFFECTED CORN FIELD NEAR SANTA ANA, CALIF.

salinity of irrigation waters and aridity, may raise salt concentrations in the soil to harmful levels, producing the condition called "salinity." Fig. 1 shows a typically salt-affected corn field. In the bare spot, salinity is prohibitively high, and only a few salt-tolerant weeds are growing. This area is surrounded by stunted plants and grades into nonsaline areas supporting good growth. Such extremes of salinity and productivity are characteristic of salt-affected land.

Before it is possible to consider the quantity of salt different plants can tolerate, it is necessary to know how different salts compare in their effects on plants. The principal ions in salt-affected soils are calcium, sodium, magnesium, chloride, sulfate, and bicarbonate. Rarely, other ions may also occur

in high concentrations. Some ions (calcium, magnesium, sulfate) provide major essential elements for plant growth; others are non-essential or required only in trace quantities. Careful experiments are necessary to determine the effects of different ions or salts on plant growth. Although plants can tolerate a range of ionic proportions in the root medium, single salt solutions may be highly toxic. A certain balance is essential, especially between monovalent cations such as sodium and divalent cations such as calcium. The effects of different salts on plant growth are, therefore, usually compared by adding the salts to a base nutrient solution in water culture or to a fertile soil. Such solutions or soils contain the required plant nutrients and also tend to balance or "antagonize" to a major degree the specific cation that may be added in high concentrations.

A common procedure in salt-tolerance studies is to add NaCl, Na_2SO_4, or $CaCl_2$ at various concentrations to a base nutrient solution. With most crop plants, these salts tend to affect growth equally when added in isosmotic concentrations.[2] (The osmotic pressure of a solution is the pressure that would have to be applied to the solution to prevent net diffusion of water into the solution when it is separated from pure water by a semi-permeable membrane.) Fig. 2 indicates the growth of bean plants in water cultures containing isosmotic concentrations of 1, 2, 3, and 4 atmospheres of indicated salts added to base nutrient water cultures. These pictures illustrate progressive growth reduction with increasing salinity and the equivalent effects of the sodium and calcium salts (top three rows). The magnesium salts, especially the $MgSO_4$, exert additional inhibitory effects.

This relationship of plant growth to osmotic pressure has an important consequence for studies on the salt tolerance of plants. It is usually unnecessary to perform tedious analyses for individual salt constituents to establish the salinity level or to evaluate the tolerance of the plant to each salt or ion present. A determination of the osmotic pressure of the soil solution generally sufficies. The electrical conductivities of soil solutions have been found to be sufficiently well related to their osmotic pressures to permit the substitution of this very simple measurement for the more involved osmotic-pressure determination. Another simplification involves the substitution of the saturation extract of the soil for the more difficultly obtainable soil solution. Thus, the electrical conductivity of the saturation extract of the soil is sufficiently well related to the osmotic pressure of the soil solution to furnish a reliable guide to the effective salinity of the soil solution.[3] The average relationship between osmotic pressure in atmospheres (OP) of the saturation extract of soil and electrical conductivity in millimhos per centimeter at 25°C ($EC_e \times 10^3$) is:

$$OP = 0.36 \left(EC_e \times 10^3\right)$$

Other useful conversion factors and equivalents include: To convert electrical conductivity of saturation extracts of soils to ppm in saturation extract, the average relationship is:

$$ppm = 640 \left(EC_e \times 10^3\right)$$

2 "Effects of High Salt Concentrations on Growth of Bean Plants," by H. G. Gauch and C. H. Wadleigh, Botanical Gazette, Vol. 105, 1944, p. 379.

3 "Diagnosis and Improvement of Saline and Alkali Soils," by U. S. Salinity Lab. Staff, U. S. Dept. of Agric. Handbook No. 60, 1954, p. 9.

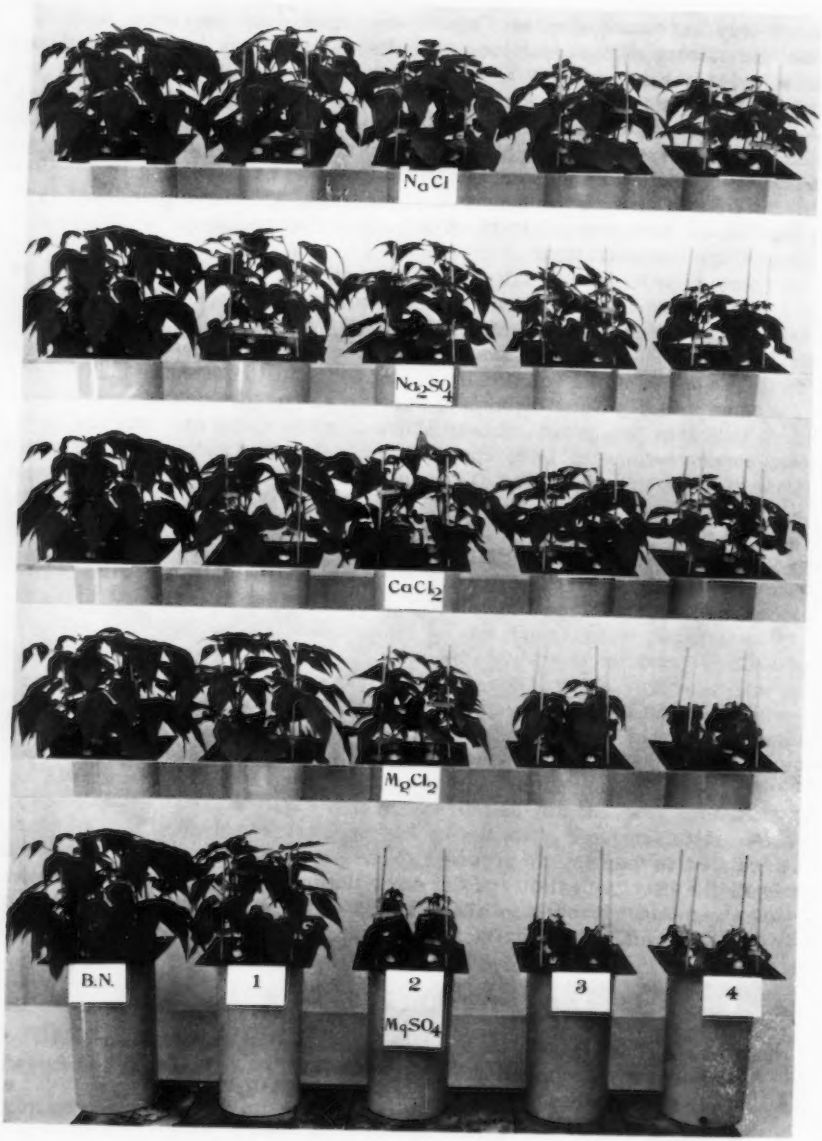

FIG. 2.—EFFECT OF ISOSMOTIC CONCENTRATIONS OF DIFFERENT SALTS ON GROWTH OF BEAN PLANTS

and to convert salt concentrations in waters expressed as grains per gallon to ppm:

$$ppm = 17.1 \times \text{grains per gallon}$$

The concentrations of single salt solutions required for one atmosphere osmotic pressure are as follows:

	meq per liter per atmosphere osmotic pressure	ppm per atmosphere osmotic pressure
NaCl	24	1403
$CaCl_2$	32	1776
Na_2SO_4	36	2557

Although osmotic relationships account for the major effect of salinity on plant growth, other factors may occasionally be important. In addition to osmotic inhibition, some ions exert specific effects that depress growth and yield still further. The specific ion effects may be toxic or nutritional in nature. An ion may be considered toxic when its presence in the solution causes direct damage to the plant. Injury usually is associated with the accumulation of harmful concentrations of the toxic ion in the plant tissues which may show no other significant change in mineral composition. The chloride and sodium ions are toxic in this sense to many fruit crops. Carefully controlled experiments have demonstrated chloride and usually also sodium toxicity in stonefruit trees (peaches, plums, apricots, and so on), avocado, citrus, strawberry blackberries, and grapes.

Moderate concentrations of chloride in the root medium (700 ppm to 1500 ppm in the soil moisture or nutrient solution) usually cause chloride to accumulate in the leaves to about 1% to 2% of the dry weight. At such concentrations, marginal leaf burn develops, leading ultimately to leaf drop, twig die-back, and even the death of the plant. Sodium accumulations in leaves of just a few tenths per cent of the dry weight produce similar leaf-burn symptoms and extensive injury. Chloride or sodium burn may be the major factor in salt damage to specifically sensitive fruit crops.[4] It must be emphasized, however, that most crops, including vegetable, field, and foreign crops, are not specifically sensitive to the chloride or sodium ions and may accumulate higher concentrations than those that injure fruit crops, without developing any specific injury symptoms. In such instances, growth depression appears to be simply a function of the osmotic pressure of the medium, regardless of the proportions of chloride or sodium. Sodium, however, may impair the physical condition of the soil and, thus, indirectly affect plant growth.

Other ions and mineral compounds may be specifically injurious to plants. Boron, although essential for plant growth at low concentrations (culture solutions usually contain 0.5 ppm B), becomes injurious at solution concentrations of only a few parts per million. Injury is usually correlated with excessive accumulation of boron in the leaves, making foliar analysis for boron

4 "Physiology of Salt Tolerance," by L. Bernstein and H. E. Hayward, Annual Review of Plant Physiology, Vol. 9, 1958, p. 25.

a very useful diagnostic tool.[5] The bicarbonate ion also tends to be specifically injurious. It interferes with normal utilization of iron by plants and may cause an iron-deficiency chlorosis. Such bicarbonate toxicity has been only rarely encountered in the field. In contrast to the widespread occurrence of salinity in irrigated regions, the specialized effects associated with boron, lithium, selenium, and other toxicities tend to be localized and are not necessarily associated with salinity.

Salinity may also inhibit growth of plants because of effects on plant nutrition. An ion present in excess may frequently have 10, 100, or even 1000 times the concentration of other ions essential for growth. Under such conditions, it is remarkable that nutritional effects, that is, deficiencies of essential nutrients, occur as infrequently as they do. Because plants vary widely in their nutrient requirements and in their ability to absorb specific nutrients, the effects of salinity on nutrition differ markedly from species to species. Thus, a given type of salinity may cause nutritional difficulties in some species but not in others. High concentrations of sulfate decrease the uptake of calcium by plants and cause some lettuce varieties to develop calcium-deficiency symptoms. Salinity also tends to increase the incidence of blossom-end rot of tomatoes, a disease likewise attributed to calcium deficiency. On the other hand, high concentrations of calcium may restrict the uptake of essential potassium. Beans and some carrot varieties are affected in this way by high concentrations of calcium. These nutritional effects are usually of secondary importance compared to the dominant osmotic effects. Moreover, most saline soils contain a mixture of soluble salts that tends to lessen or eliminate the harmful effects resulting from a great preponderance of a single salt.

From all of the foregoing, it follows that tolerance of most plants to salinity can be adequately described by reference to the total soluble-salt content in the root zone measured as the electrical conductivity of the saturation extract of the soil. Except for the most tolerant crops, which are not damaged by low or moderate salinities, most plants show a progressive decline in growth and yield with increasing salinity. For such crops, there is no completely "safe" limit of salinity, but an upper limit that still permits economic yields may be given.[6,7,8] The most tolerant crops include some of the grasses such as Bermudagrass and tall wheatgrass, and also barley, sugar beets, and cotton. These crops can tolerate soil salinities in excess of 8 millimhos per cm and still yield well. Some varieties of Bermudagrass, for example, are producing full yields in experimental plots having electrical conductivities of 15 mmhos per cm. The yields of most crops, however, fall off seriously at conductivities less than 8 mmhos per cm. Tomato, one of the more tolerant vegetable crops, declines appreciably in yield at conductivities greater than 5 mmhos per cm. Alfalfa has a comparable level of salt tolerance, whereas most true clovers are quite sensitive to salinity and can tolerate no more than about 2 mmhos per cm. Beans and most fruit crops are similarly salt-

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5 "Boron Injury to Plants," by L. V. Wilcox, Agriculture Information Bulletin No. 211, U. S. Dept. of Agric., 1960.

6 "Salt Tolerance of Grasses and Forage Legumes," by L. Bernstein, Agriculture Information Bulletin No. 194, 1958.

7 "The Salt Tolerance of Vegetable Crops in the West," by L. Bernstein, Agriculture Information Bulletin No. 205, 1959.

8 "The Salt Tolerance of Field Crops," by L. Bernstein, Agriculture Information Bulletin No. 217, 1959.

sensitive. The general relationship of crop response to the electrical conductivity of saturation extracts of soils is given in Table 1.

Although different varieties of a given crop are usually similar in salt tolerance, with some crops, such as barley, significant varietal differences have been observed. Rootstocks for fruit trees and vines may differ markedly in their uptake of chloride, sodium, and boron. The use of salt-resistant rootstocks for citrus, stone fruits, avocado, and grape may significantly improve their salt tolerance.

Qualitatively, the effects of salinity are usually not very distinctive, except for the leaf-burn patterns associated with specific-ion toxicities. Salt-affected plants tend to be stunted and bear smaller leaves of a darker green color than normal. Heavier waxy coatings on the leaves often give the foliage a bluish-green cast. Salt injury of this type may often escape notice, even though yield losses may be as high as 20% or 30%.

Plants may be more sensitive to salinity at one stage of growth than at another. For example, sugar beets during germination are affected by salinity in the order of 4 mmhos per cm to 6 mmhos per cm (electrical conductivity of the saturation extract) but can tolerate two or three times as much salt once the plants are established. Although other crops are not notably more

TABLE 1.—CROP YIELDS AT DIFFERENT SALINITY LEVELS

Salinity effects mostly negligible	Yields of very sensitive crops may be restricted	Yields of many crops restricted	Only tolerant crops yield satisfactorily	Only a few very tolerant crops yield satisfactorily
0	2	4	8	16

Scale of electrical conductivity of saturation extracts in millimhos per centimeter at 25° C.

sensitive during germination than at later stages, germination is frequently adversely affected because salt tends to accumulate in the surface layers of soil as a result of evaporation. Furrow irrigation greatly aggravates salt accumulation so that special planting, or irrigation practices, or both, may often be required to produce satisfactory stands on even slightly saline soils.

Many crops are more sensitive during the seedling stage than at later growth stages. Rice can germinate at salinities up to 10 mmhos per cm or 15 mmhos per cm but the plants usually die if the salinity is in excess of 5 mmhos per cm or 6 mmhos per cm during the seedling stage. Rice is also quite sensitive during fruiting, as indicated by the fact that grain production may be reduced by more than 50% when straw yield has hardly been affected at all. In contrast, barley and cotton can be quite stunted, vegetatively, but still produce normal yields of grain and fiber, respectively. Barley, like rice, is more sensitive to salinity during the seedling stage than at earlier or later growth stages. Special practices may sometimes be required to permit the crop to survive during phases of minimum salt tolerance. Thus, the paddy field is sometimes drained and refilled with fresh water, to lower salinity during the critically sensitive flowering stage of rice.

Climatic factors may also modify the salt tolerance of plants. A very striking example is the much greater sensitivity of onions in hot, dry inland

valleys of California than at cooler, more humid coastal locations. However, other crops studied did not show as great a locational effect as onions. Fig. 3 indicates the interaction between climate and salinity on yield of onions.[9] Under nonsaline conditions (top row), onions yielded equally well at all three locations. Salinity of about 4 atmospheres (lower three rows) was much more inhibitory in warmer climates of Riverside and especially Indio than at the cooler, more humid coastal location of Torrey Pines. The moisture regime of soils may also profoundly modify plant response to a given salinity level.

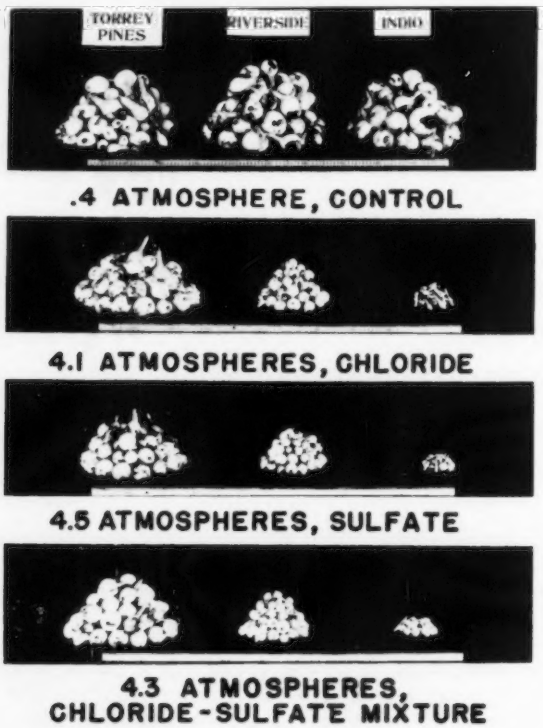

FIG. 3.—INTERACTIVE EFFECTS OF CLIMATE AND SALINITY ON YIELD OF ONIONS

The osmotic pressure of the soil solution has been shown to be the major factor that governs plant response. Immediately following an irrigation, the soil moisture content is maximal, and the soluble salt concentration is at a minimum. As water is lost from the soil by evaporation and plant use (transpira-

9 "Effect of Salt Concentration, Kind of Salt, and Climate on Plant Growth in Sand Culture," by O. C. Magistad, A. D. Ayers, C. H. Wadleigh, and H. G. Gauch, Plant Physiology, Vol. 18, 1943, p. 151.

tion), the salts tend mostly to be left behind in a shrinking volume of water. The salt concentration, therefore, increases progressively, and the average concentration over an irrigation cycle is higher, the drier the soil is allowed to become before re-irrigation. More frequent irrigation, by preventing excessive concentration of the soil solution, usually tends to minimize the harmful effect of a given salinity level, as illustrated in Fig. 4 which shows the interactive effects of salinity and irrigation regime on yield of beans.[10] Increasing salinity (from left to right) is more detrimental with infrequent irrigation (medium and high tension) than with more frequent irrigation (low tension).

Because the irrigation water is usually a primary source of salt, each irrigation tends to add another increment of salinity to the soil. If only enough water is added to re-moisten the root zone without any percolation into deeper soil layers, the salt additions will simply cause step-wise increases in the salinity of the root zone. An excess of irrigation water, to keep the salt moving through the soil and beyond the root zone, is a prime requisite for continued irrigation. Saline soils, therefore, not only require irrigation at a higher residual moisture content than do nonsaline soils but also require the application of extra water to allow for removal of salt from the root zone.

The proportions of adsorbed cations in the soil may be as important as the concentration of soluble salts. With regard to the adsorbed or exchangeable cations, only the percentage of exchangeable sodium is of direct concern in salt-affected soils. Sodic soils, by definition, are those that contain 15% or more of exchangeable sodium (that is 15% or more of the total exchangeable cations on a chemical equivalent basis are sodium). At such exchangeable sodium percentages, the physical properties of soils generally deteriorate, so that growth of almost any plant is adversely affected. Fig. 5 shows that sodium-tolerant beet plants produce normal yields at exchangeable-sodium-percentages (ESP) up to 50 when a favorable soil condition is maintained[11] (PK series, Pachappa soil plus Krilium, a soil conditioner). In the absence of soil conditioner (P series), yield is already depressed at 25% exchangeable sodium. Some plants, such as bean, are particularly sensitive to exchangeable sodium and may suffer adverse nutritional effects, even though the physical condition of the soil is still favorable (at exchangeable-sodium-percentages of about 10). Finally, many of the fruit crops are specifically sensitive to sodium and develop severe injury symptoms when the exchangeable-sodium-percentage[12] is only about 5%.

Many of the factors that must be considered in relation to the salt tolerance of plants have been enumerated and described. Condensed in so brief a space, the problem may appear exceedingly complex and not too well-defined. Yet, usable tolerance levels can be given for most crop plants.[6,7,8] These values, however, refer to permissible concentrations in the soil solution (or the saturation extract) and not to permissible concentrations in the irrigation water. A soil equilibrated with a given water will have an electrical conduc-

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10 "The Interrelationships of Salt Concentration and Soil Moisture Content with the Growth of Beans," by A. D. Ayers, C. H. Wadleigh, and O. C. Magistad, Journal, Amer. Soc. of Agronomy, Vol. 35, 1943, p. 796.

11 "The Influence of Exchangeable Sodium on the Yield and Composition of Plants—Green Beans, Garden Beets, Clover and Alfalfa," by L. Bernstein and G. A. Pearson, Soil Science, Vol. 82, 1956, p. 247.

12 "Tolerance of Crops to Exchangeable Sodium," by G. A. Pearson, Agriculture Information Bulletin No. 216, 1959.

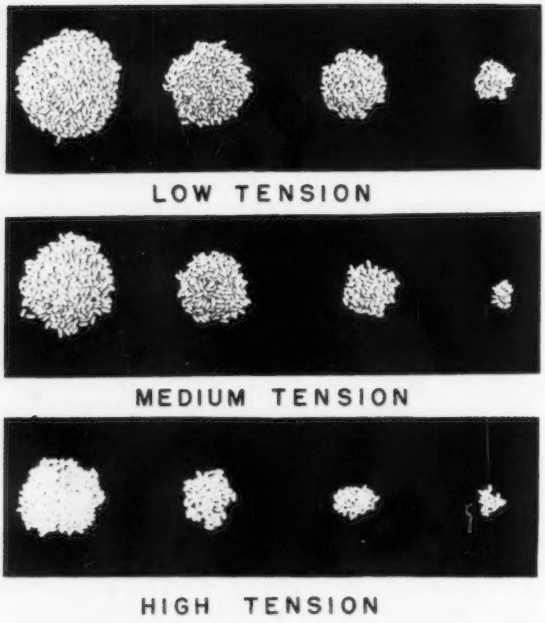

FIG. 4.—INTERACTIVE EFFECTS OF SALINITY AND IRRIGATION REGIME ON YIELD OF BEANS

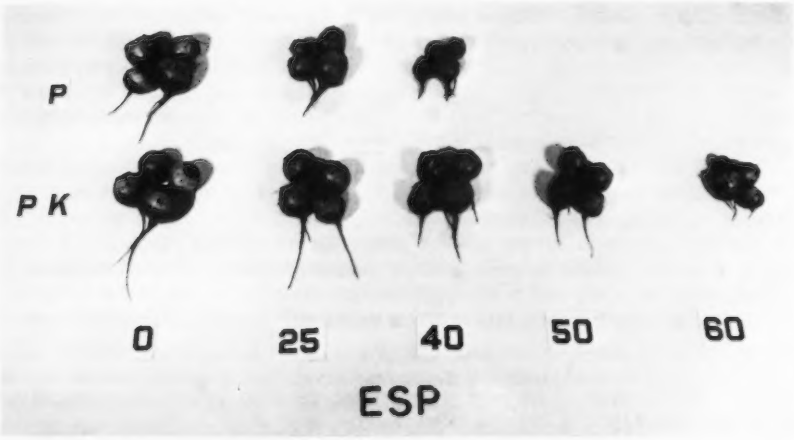

FIG. 5.—EFFECTS OF EXCHANGEABLE SODIUM PERCENTAGE ON YIELDS OF GARDEN BEETS IN THE PRESENCE AND ABSENCE OF KRILIUM, A SOIL CONDITIONER

tivity of the saturation extract (EC_e) approximately one-half that of the electrical conductivity (EC) of the applied water, because the water content at the saturation percentage is approximately twice that at field capacity.

Although this relationship holds under ideal conditions of irrigation and drainage, the EC_e of most soils is several to many times the minimal (EC_e) associated with ideal conditions. Moreover, because drainage conditions, irrigation practices, and other factors, such as rainfall, are so variable, the suitability of a given water for irrigation can be assessed only if all the conditions under which it is to be used are carefully considered. Under average conditions, a moderate to large salt accumulation in the soil must be allowed for.

Thus, it is seen that moderately tolerant crops may be affected by salinities as low as 3 mmhos per cm (EC_e). A water with an EC of 6 mmhos per cm should produce an EC_e of 3 mmhos per cm in the absence of salt build-up. For such a crop, however, irrigation with water having a conductivity in excess of 1 mmho per cm would usually be considered too hazardous.

This gives some idea of the ratio of average to minimal soil salinities associated with a given irrigation water. A factor of 5- or even 10-fold accumulation is not at all unusual. Of course, more nearly ideal conditions are occasionally encountered. There are areas in which irrigation with waters having an EC of 5 mmhos per cm or even 10 mmhos per cm is regularly practiced. In these instances, only salt-tolerant crops are grown (grasses as seed crops, for example), and the salt-accumulation factor is evidently kept at nearly minimum values by appropriate management practices and favorable soil characteristics.

Entirely analogous arguments apply for the concentration of specific ions. If, for a given crop, 40 meq per l in the solution bathing the roots is indicated as the maximum tolerable level, irrigation with water having more than 5 meq per l for example, may be considered as too hazardous under average conitions. But irrigation with waters having 10 meq per l or even 20 meq per l under more favorable conditions and with careful management is not necessarily precluded.

Plant response to salinity depends on the total concentration, and sometimes the proportions, of ions in the soil solution. It may vary with the stage of development for a given plant or be modified by climatic factors and by irrigation regimes but it is, nonetheless assessable and, in most instances, sufficiently predictable. The relationship of soil salinity to irrigation-water quality is likewise controlled by a number of factors. Some of these, such as adequacy of irrigation (allowance for leaching requirement), may be controlled within limits. Often, however, poor soil permeability and drainage may place severe restrictions on the quantity of water that may be applied, thus limiting the approach to minimal salt levels in the soil. In any event, the spectrum of soil characteristics and management practices that affect salt accumulation under field conditions is so broad and difficult to define that this becomes the weakest link in attempting to relate plant response directly to irrigation water quality.

CONCLUSIONS

1. Plant response to salinity is governed primarily by the osmotic pressure of the soil solution, although specific ion effects may be important for some crops under certain conditions.

2. Inhibition of growth by salinity can often be reduced by more frequent irrigation, that is, by keeping the soil moisture at a higher level. Saline soils generally should be irrigated at higher residual soil moisture levels than comparable nonsaline soils.

3. Different varieties of a given crop usually have similar salt tolerance; a few exceptions are noted. Some rootstocks or varieties of fruit crops can restrict the uptake of ions (chloride, sodium, borate) to which these crops are specifically sensitive.

4. Irrigation with saline waters requires the application of extra quantities of water to promote leaching of salt through the soil, and to prevent a progressive accumulation of salt in the root zone.

5. Crop response to salinity can be adequately described in terms of the electrical conductivity of saturation extracts of soils and the concentration of toxic ions in the root zone. Because the level of salt accumulation in the soil from the use of a given irrigation water can be quite variable, it is not possible to predict crop response to an irrigation water without taking into account the conditions under which it is to be used.

ACKNOWLEDGMENTS

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UNIFICATION OF PARSHALL FLUME DATA

By Sydney Davis,[1] M. ASCE

SYNOPSIS

The Parshall critical depth flume has been used extensively in the Western United States and other parts of the world since about 1925 and is accepted as a standard measuring device in open channel irrigation.

The original design, which was somewhat empirical, has been maintained until now; also the results have been presented as a number of empirical formulae covering widths ranging from 1 in. to 50 ft. For these reasons engineers have hesitated to use sizes of flumes other than those specified and have felt that great care must be taken to build the flumes according to the dimensions given.

In this paper, dimensional methods have been used to develop a semi-theoretical equation relating flow and depth for all flumes from 1 in. to 50 ft. Excellent agreement between this equation and all published data is found. This will permit using flumes of non-standard sizes and will broaden the field of application for this type of measuring device.

INTRODUCTION

The Parshall flume as used in 1961 is based on experiments that were continued over several years. The results of the first experiments for flumes of

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[1] Visiting Assoc. Prof., Cornell Univ., Ithaca, N. Y.; Senior Lecturer, Israel Inst. of Tech., Haifa, Israel.

throat widths 1 ft to 8 ft were presented[2] in 1926. Subsequently, data were given[3,4] for flumes of sizes 10 ft to 50 ft, 3 in., 6 in. and 9 in. and in[5] 1957 1 in. and 2 in. All the data was presented as empirical formulas, seven in number, of the form

$$Q = K_e\, b\, y_1^n \qquad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

with the values of K_e and n are given in Table 1. In Eq. 1 Q refers to the discharge, K_e is a discharge coefficient to be defined, b denotes the channel width, and y_1 is the depth at measuring section. The "inch" flumes were constructed of steel, the 1 ft to 8 ft flumes, of wood and the largest 10 ft to 50 ft flumes of concrete.

The flume dimensions were chosen in an arbitrary manner. Referring to Fig. 1, all flumes have the same side angle θ - $\tan^{-1}$ 0.2, and dropdown angle ϕ - $\tan^{-1}$ 0.375. The other important dimension as far as this paper is concerned is x_1, the centerline distance from the throat crest to the upstream

TABLE 1.—VALUES OF K_e AND n IN EQ. 1

Throat Width	K_e	n
1 in.	4.06	1.55
2 in.	4.06	1.55
3 in.	3.97	1.547
6 in.	4.12	1.580
9 in.	4.09	1.530
1 ft to 8 ft	4	$1.522\, b^{0.026}$
10 ft to 50 ft	$3.6875 + \frac{2.5}{b}$	1.6

measuring section. For the "foot" flumes this is related to the throat width by the equations,

$$x_1 = 0.327\, b + 2.610 \qquad \ldots\ldots\ldots\ldots\ldots \quad (2a)$$

and

$$b_1 = b + 0.4\, x_1 = 1.131\, b + 0.653 \qquad \ldots\ldots\ldots\ldots \quad (2b)$$

For the "inch" flumes there are no corresponding relationships.

The empirical nature of this work has discouraged theoretical analysis, and such attempts as have been made seem to have been unsuccessful.[6] For the

2 "The Improved Venturi Flume," by Ralph L. Parshall, Transactions, ASCE, Vol. 89, 1926, p. 841.

3 "Parshall Flumes of Large Size," by Ralph L. Parshall, Colo. Agric. Experiment Sta., Bulletin No. 386, May, 1932.

4 "The Parshall Measuring Flume," by Ralph L. Parshall, Colo. Agric. Experimental Sta., Bulletin No. 423, March, 1936.

5 "Parshall Measuring Flumes of Small Sizes," by A. R. Robinson, Tech. Bulletin No. 61, Agric. Experimental Sta., Fort Collins, Colo., October, 1957.

6 "Critical Flow Meters (Venturi Flumes)," by A. Balloffet, Proc. Paper No. 743, ASCE, Vol. 81, July, 1955.

same reason it is necessary to keep rigidly to the specified dimensions,[7] which has obvious disadvantages. In this paper the seven empirical formulas are reduced to one comprehensive formula which, as will be explained, gives a degree of flexibility in the choice and design at the flume.

All calculations are based on original data which differ slightly from those published in tables.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in the Appendix.

THEORY

The portion of a Parshall flume between the upstream measuring point and the throat is a transition section with straight converging walls and a horizontal floor. However, the hydraulics of the flume is complicated by the dropdown at the throat causing curvature of the stream lines and a resulting reduction in pressure below hydrostatic at and near the throat.

Research carried out on the free overfall in parallel channels[8,9] shows the pressure reduction causes the critical depth for hydrostatic flow to occur upstream of the drop at a section at which the pressure is approximately hydrostatic. The critical depth is given by

$$y_{cn} = \sqrt[3]{\frac{Q^2}{b_{cn}^2 g}} \qquad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

in which b_{cn} is the channel width at which critical depth occurs, Q denotes the discharge, and g is the acceleration due to gravity. In the present case, a similar effect may be expected but with the complication that y_{cn} occurs at a section of unknown width b_{cn}. This has been shown experimentally by V. L. Hauser,[10] (See also Fig. 1). The problem is made more difficult by possible frictional effects near the throat where the velocity is very high.

Neglecting scale effect it may be assumed that the variables involved in the Parshall flume are: Q, b, y_1, x_1, g, θ, and ϕ (See Fig. 1) in which y_1 is the depth at distance x_1 from the throat crest, that is

$$f_1 (Q, b, y_1, x_1, g, \theta, \phi) = 0 \qquad \ldots\ldots\ldots\ldots \quad (4)$$

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7 "Water Measurement Manual," Bur. of Reclamation, May, 1953.

8 "Analyzing Hydraulic Models for Effect of Distortion," by M.P. O'Brien, Engineering News-Record, McGraw-Hill Publishing Co., Inc., New York, Vol. 109, September 15, 1932.

9 "Discharge Characteristics of the Free Overfall," by Hunter Rouse, Civil Engineering, Vol. 6, No. 4, 1936.

10 "The Relationship Between the Length of the Converging Section and the Free Flow Equation for the Venturi Flume," by V. L. Hauser, thesis presented to the Univ. of California, in Davis, Calif., in 1957, in partial fulfilment of the requirements for the degree of Master of Science.

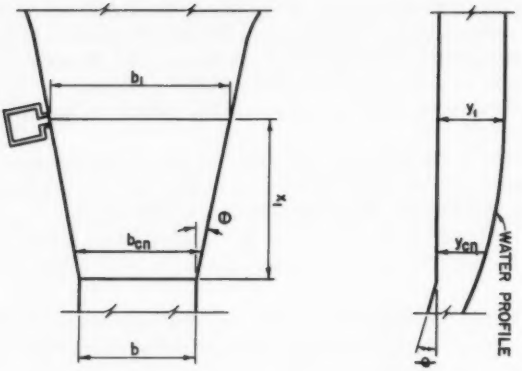

FIG. 1

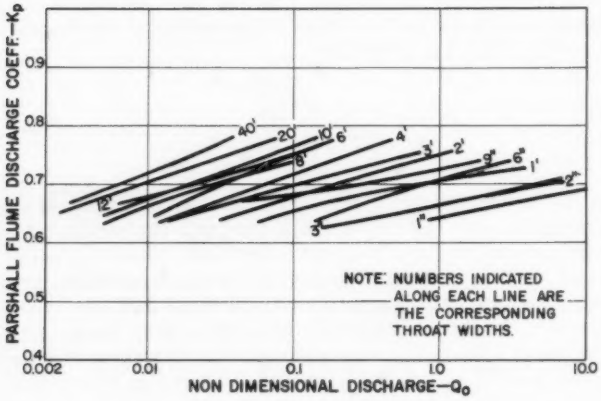

FIG. 2

By dimensional analysis Eq. 4 may be reduced to

$$\frac{Q}{g^{1/2}\, b\, y_1^{3/2}} = f_2\left(\frac{Q}{g^{1/2}\, b^{5/2}},\ \frac{x_1}{b},\ \theta,\ \phi\right) \quad \ldots\ldots\ldots \quad (5)$$

For Parshall flumes θ and ϕ are constant, therefore,

$$\frac{Q}{g^{1/2}\, b\, y_1^{3/2}} = f_3\ (Q_o,\ x_o) \quad \ldots\ldots\ldots\ldots \quad (6)$$

in which

$$Q_o = \frac{Q}{g^{1/2}\, b^{3/2}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

and

$$x_o = \frac{x_1}{b} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

in which f_2 and f_3 are unknown functions, so that

$$\frac{Q}{g^{1/2}\, b\, y_1^{3/2}} = K_p \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

in which K_p is a discharge coefficient.

Using the published data, K_p has been calculated and plotted against Q_o for various flume widths or for what amounts to the same thing, for various values of x_o, because x_o is different for each flume (see Fig. 2).

It is seen that not only does K_p differ for each flume, but there is a variation with Q_o for the same flume. The reason for the variation will be explained subsequently.

Using dimensional analysis once again, Eq. 4 may be expressed as

$$\frac{Q}{g^{1/2}\, b^{5/2}} = f_4\left(\frac{y_1}{b}, \frac{x_1}{b}\right) \qquad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

or

$$Q_o = f_4\,(x_o\, y_o) \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which

$$y_o = \frac{y_1}{b} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

If two geometrically similar flumes A and B were being compared, then x_{oA} and x_{oB} would be equal and Eq. 11 implies that if the depths at the measuring section are such that $y_{oA} = y_{oB}$ then $Q_{oA} = Q_{oB}$.

In general each different size Parshall flume has a different value of x_o. Fortunately, Hauser has measured the flow profiles in a 1 ft flume for four different flows. He used piezometers for measuring the depths, so it should be possible to compare these results with Parshall's which were obtained by the same method.

The flow profiles are shown plotted (non-dimensionally) in Fig. 3, each curve corresponding to the non-dimensional flow parameter Q_o. In addition, values of x_o and y_o have been calculated for each Q_o from different size flumes; a plot of these values is also shown.

Example.—Consider $Q_o = 0.0666$. For the 20 ft flume this corresponds to an actual flow of

$$(0.0666)\ (g^{1/2})\ (20^{5/2}) = 677 \text{ cfs}$$

From published data[3] the depth y_1 corresponding to this flow is 3.9 ft, therefore,

$$y_o = \frac{y_1}{b} = \frac{3.9}{20} = 0.195$$

Also for this size flume x = 9.15 ft, therefore,

$$x_o = \frac{x_1}{b} = \frac{9.15}{20} = 0.4575$$

It may be seen from the plot that any particular value of Q_o does not cover the full range of flume sizes.

The excellent agreement between the derived points and the experimental plotting suggests a further step. If, at any measuring section, parallel uniform flow is assumed, then:

$$E_1 = y_1 + \frac{Q_1^2}{2\,g\,y_1^2\,b_1^2} \qquad \text{.....(13a)}$$

or

$$E_1 = y_1 + \frac{Q_1^2}{2\,g\,y_1^2\,(b+0.4\,x_1)^2} \qquad \text{.....(13b)}$$

in which E_1 is the specific energy head. Dividing by b yields

$$\frac{E_1}{b} = \frac{y_1}{b} + \frac{Q_1^2\,(g\,b^5)}{2\left(\frac{y_1}{b}\right)^2\left(1+0.4\,\frac{x_1}{b}\right)^2} \qquad \text{.....(14a)}$$

or

$$E_o = y_o + \frac{Q_o^2}{2\,y_o^2\,(1+0.4\,x_o)^2} \qquad \text{.....(14b)}$$

For each value of Q_o, E_o was calculated for the various values of x_o and y_o. It is seen from Table 2 that E_o is almost constant for each Q_o which means that energy losses in the flumes between $0.392 < x_o < 9.5$ are negligible and that any appreciable loss that may occur must be close to the throat. The non-dimensional energy head was further calculated for flows and flumes of all sizes. The resulting plot of E_o versus Q_o on log-log paper (Fig. 4) produced a straight line whose equation was found to be

$$E_o = 1.351\ Q_o^{0.645} \qquad \text{.....(15)}$$

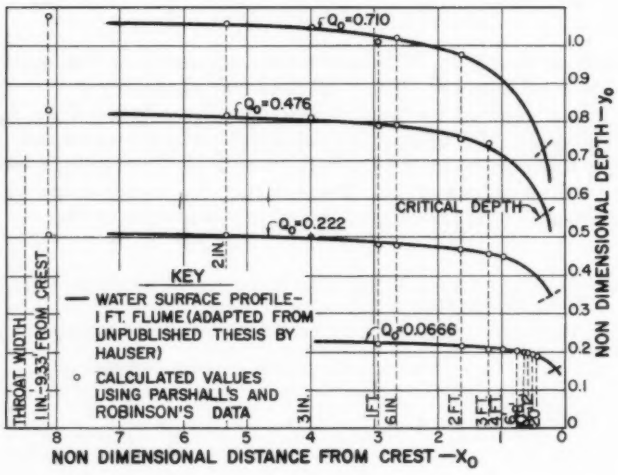
Q_0=0.710
Q_0=0.476
Q_0=0.222
Q_0=0.0666
CRITICAL DEPTH
KEY
— WATER SURFACE PROFILE-1 FT. FLUME (ADAPTED FROM UNPUBLISHED THESIS BY HAUSER)
○ CALCULATED VALUES USING PARSHALL'S AND ROBINSON'S DATA
THROAT WIDTH 1 IN.-9.33" FROM CREST
2 IN.
3 IN.
1 FT.
6 IN.
2 FT.
3 FT.
4 FT.
6 FT.
8 FT.
20 FT.
NON DIMENSIONAL DEPTH — y_0
NON DIMENSIONAL DISTANCE FROM CREST — X_0

FIG. 3

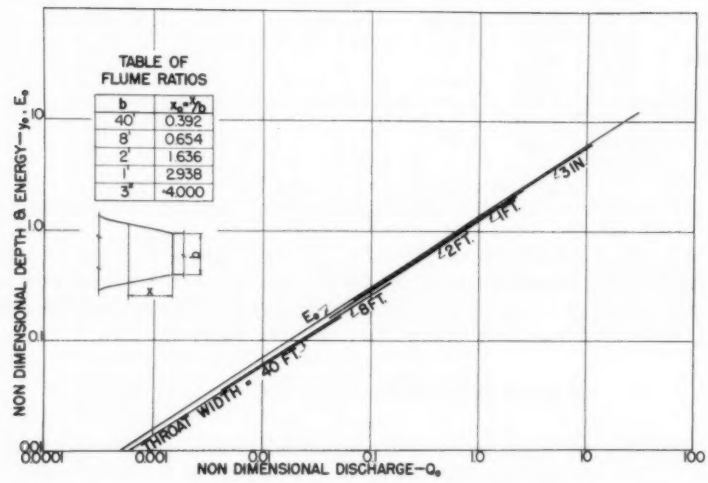
TABLE OF FLUME RATIOS
b | $x_0=x/b$
40' | 0.392
8' | 0.654
2' | 1.636
1' | 2.938
3" | 4.000
NON DIMENSIONAL DEPTH & ENERGY — y_0, E_0
NON DIMENSIONAL DISCHARGE — Q_0
THROAT WIDTH = 40 FT.
8 FT.
2 FT.
1 FT.
3 IN.
$E_0=y_0$

FIG. 4

or

$$y_o + \frac{Q_o^2}{2\,y_o^2\,(1+0.4\,x_o)^2} = 1.351\,Q_o^{0.645} \quad \ldots\ldots\ldots \quad (16)$$

(The points from which Eq. 15 is derived are not shown in Fig. 4; almost without exception the points fall within the thickness of the line.)

Eq. 16 is the general Parshall flume equation relating Q, y, b, and x for flumes of any size. The solution to this equation is laborious, but it has been solved using a computer at the Cornell University Computing Center, Ithaca, N, Y., for various values of x_o. The solution for some values of x_o corresponding to flumes of standard widths are shown plotted in Fig. 4.

TABLE 2.—VALUES OF NON-DIMENSIONAL FLOW, DEPTH AND ENERGY FOR DIFFERENT SIZES OF FLUME

Q_o		0.0666		0.222		0.476		0.710	
b	x_o	y_o	E_o	y_o	E_o	y_o	E_o	y_o	E_o
1 in.	9.31	–	–	0.510	0.514	0.829	0.836	1.08	1.089
2 in.	5.33	–	–	0.500	0.510	0.816	0.833	1.058	1.080
3 in.	4.00	–	–	0.500	0.515	0.811	0.836	1.050	1.084
6 in.	2.52	–	–	0.487	0.511	0.790	0.832	1.020	1.078
9 in.	2.67	–	–	0.485	0.511	0.764	0.812	0.995	1.057
1 ft	2.94	0.220	0.230	0.470	0.494	0.780	0.829	1.01	1.072
2 ft	1.64	0.215	0.233	0.465	0.507	0.755	0.827	0.975	1.070
3 ft	1.20	0.211	0.234	0.444	0.501	0.744	0.837	0.960	1.080
4 ft	0.983	0.208	0.234	0.440	0.505	–	–	–	–
6 ft	0.763	0.206	0.237	–	–	–	–	–	–
8 ft	0.654	0.205	0.238	–	–	–	–	–	–
10 ft	0.589	0.201	0.237	–	–	–	–	–	–
12 ft	0.546	0.199	0.237	–	–	–	–	–	–
20 ft	0.458	0.195	0.235	–	–	–	–	–	–
40 ft	0.392	–	–	–	–	–	–	–	–
		Mean = 0.235			0.508		0.833		1.077

Comparisons between the solution of Eq. 16 and the original data show agreement with an error of less than 1% for the majority of points and less than 2% for the rest with some very few exceptions (see Table 3).

DETERMINATION OF THEORETICAL FLOW

Considering the flume as an open channel transition, assume (1) flow at the measuring section uniform and parallel, and (2) flow at the throat non-uniform and curved. Having obtained a relationship for the actual flow rate, a theoretical expression will now be derived. Referring to Fig. 1

$$E_1 = y_1 + \frac{V_1^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (17a)$$

TABLE 3.—ACCURACY OF EQ. 16

Flume Size	Error, in percentage	Remarks
1 in.	< 1	
2 in.	< 1	
3 in.	< 1	< - 2% large flows
6 in.	< 1	< + 2% large flows
9 in.	Approx. 5	Original calibration suspected
1 ft	- 2 to 3	Scatter of original data.[a]
2 ft	2	Scatter of original data.[a]
3 ft	< 1	
4 ft	< 1	
6 ft	< 1	< 2% small flows
8 ft	< 1	
10 ft	< 1	
12 ft	< 2	Scatter of original data.
20 ft	< 1	2% small flows
40 ft	< 1	

[a] Parshall reported trouble with ice at time of calibration.

in which E_1 is total energy head at the measuring point x_1, and V_1 is the mean velocity at the measuring section, or

$$\frac{g\,E_1}{V_1^2} = \frac{1}{2\left(1 - \frac{y_1}{E_1}\right)} \quad \ldots\ldots\ldots\ldots\ldots (17b)$$

also

$$y_c = \lambda_1 \frac{2}{3} E_1 \quad \ldots\ldots\ldots\ldots\ldots (18)$$

and

$$V_c = \lambda_2 \sqrt{g\,y_c} \quad \ldots\ldots\ldots\ldots\ldots (19)$$

in which λ_1 and λ_2 are factors correcting for non-hydrostatic pressure, friction and non-uniform velocity distribution, and V_c is the critical velocity. Substituting for y_c from Eq. 18, Eq. 19 becomes

$$V_c = \lambda_2\, \lambda_1^{\frac{1}{2}} \sqrt{\frac{2}{3} g\, E_1} \quad \ldots\ldots\ldots\ldots (20)$$

Also

$$Q = b\, V_c\, y_c \quad \ldots\ldots\ldots\ldots\ldots (21)$$

Substituting from Eqs. 18 and 20 and writing $\lambda_2\, \lambda_1^{3/2} = \lambda$

$$Q = \lambda \left(\frac{2}{3}\right)^{\frac{3}{2}} b\, g^{1/2}\, E_1^{3/2} \quad \ldots\ldots\ldots\ldots (22a)$$

or

$$Q = \lambda \frac{\left(\frac{2}{3}\right)^{\frac{3}{2}} b\, g^{1/2}\, y_1^{3/2}}{\left(\frac{y_1}{E_1}\right)^{3/2}} \quad \ldots\ldots\ldots\ldots (22b)$$

Comparing with Eq. 9

$$K_p = \frac{\lambda\, (2/3)^{3/2}}{\left(\frac{y_1}{E_1}\right)^{3/2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The value of λ may be found by writing Eq. 22 as

$$Q_o = \lambda \left(\frac{2}{3}\right)^{\frac{3}{2}} E_o^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

and comparing with Eq. 15

$$\lambda = 1.17\, Q_o^{0.0325} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Eq. 23 then becomes

$$K_p = 1.17\, Q_o^{0.0325} \frac{(2/3)^{3/2}}{\left(\frac{y_1}{E_1}\right)^{3/2}} \quad \ldots\ldots\ldots\ldots (26)$$

It is thus seen that K_p is a function of $\frac{y_1}{E_1}$ and Q_o $\frac{y_1}{E_1}$ can be expressed as a function of β and Q_o as follows:

From continuity

$$V_1\, y_1\, b_1 = V_c\, y_c\, b \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

therefore

$$y_1 = \frac{V_c}{V_1}\, y_c\, \frac{b}{b_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

or

$$y_1 = \frac{V_c}{V_1}\; y_c\, \beta \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Substituting in Eq. 29 from Eqs. 19, 18, and 17b and rearranging gives

$$\left(1 - \frac{y_1}{E_1}\right) \left(\frac{y_1}{E_1}\right)^2 = \frac{4}{27} \lambda^2\, \beta^2 \quad \ldots\ldots\ldots\ldots (30)$$

Solving the cubic equation yields, for the relevant value,

$$\frac{y_1}{E_1} = \frac{2}{3} \cos \left[1/3 \cos^{-1} \left(1-2 \lambda^2 \beta^2\right)\right] + 1/3 \quad \ldots\ldots\ldots (31)$$

and eliminating $\frac{y_1}{E_1}$ from Eq. 23

$$K_p = \frac{\lambda}{\left\{\cos \left[1/3 \cos^{-1} \left(1 - 2 \lambda^2 \beta^2\right)\right] + 1/2\right\}^{3/2}} \quad \ldots\ldots\ldots (32)$$

Because λ is a function of Q_o, it is seen that K_p depends on both β and Q_o. β changes from flume to flume, depending only on the geometry, while Q_o is a function of discharge and flume width. This accounts for the variation of K_p shown in Fig. 2.

Note that for a parallel throated flume with friction neglected $\lambda = 1$ then, Eq. 30 becomes

$$\left(1 - \frac{y_1}{E_1}\right) \left(\frac{y_1}{E_1}\right)^2 = \frac{4}{27} \beta^2 \quad \ldots\ldots\ldots\ldots (33)$$

and Eq. 23 becomes

$$K = \frac{(2/3)^{3/2}}{\left(\frac{y_1}{E_1}\right)^{3/2}} \quad \ldots\ldots\ldots\ldots\ldots (34)$$

The relationship between β, $\frac{y_1}{E_1}$ and K as given by Eqs. 33 and 34 are shown plotted in Fig. 5.

Also Eq. 32 becomes

$$K = \frac{1}{\left\{\cos \left[1/3 \cos^{-1} \left(1 - 2 \beta^2\right)\right] + 1/2\right\}^{3/2}} \quad \ldots\ldots\ldots (35)$$

It will now be shown that λ is connected with the location of the nominal critical depth y_{cn}.

Dividing both sides of Eq. 3 by b yields

$$\frac{y_{cn}}{b} = \sqrt[3]{\frac{Q_o^2}{\left(\frac{b_{cn}}{b}\right)^2}} \quad \ldots\ldots\ldots\ldots\ldots (36a)$$

or

$$\frac{b_{cn}}{b} = \frac{Q_o}{\left(\frac{y_{cn}}{b}\right)^{3/2}} \quad \ldots\ldots\ldots\ldots\ldots (36b)$$

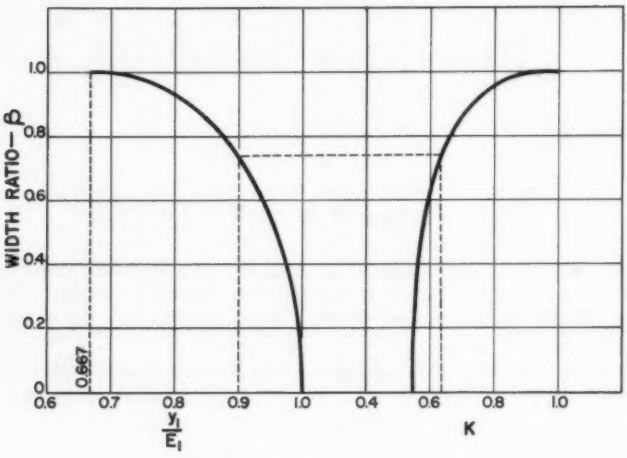

FIG. 5

Referring once again to Fig. 3 the critical depth y_{cn} for each flow profile was estimated by Hauser[10] using Eq. 3. Because b_{cn} was unknown, this was done by a trial process. The locations of y_{cn} are shown in the figure at the intersections of the flow profile and the oblique broken lines.

Using these four values the relationship between $\frac{y_{cn}}{b}$ and Q_o is found to be

$$\frac{y_{cn}}{b} = 0.900\, Q_o^{0.645} \quad \ldots\ldots\ldots\ldots\ldots \quad (37)$$

This checks with Eq. 15 for, neglecting friction loss, obviously

$$\frac{y_{cn}}{b} = 2/3\, E_o \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (38)$$

Substituting Eq. 37 into Eq. 36 gives

$$\frac{b_{cn}}{b} = 1.17\, Q_o^{0.0325} \quad \ldots\ldots\ldots\ldots\ldots \quad (39)$$

that is

$$\lambda = \frac{b_{cn}}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (40)$$

It is thus seen that the variation of the Parshall coefficient is due, at any rate for the range of Eq. 36 (Q_o = 0.0666 - 0.710), to the hydrostatic critical depth occurring upstream of the throat and that friction losses are negligible up to this section.

For smaller values of Q_o Eq. 36 may not apply because friction becomes more appreciable as Q_o diminishes, however further experiments are needed to verify this.

CONCLUSIONS

Eq. 16

$$y_o + \frac{Q_o^2}{2\, y_o^2\, (1+0.4\, x_o)^2} = 1.351\, Q_o^{0.645} \quad \ldots\ldots\ldots (16)$$

closely fits all data published on Parshall flumes and may be used for calculating the flow for flumes of any size, not only those published in the literature but also for intermediate sizes. It may also be used for correcting the calibration curves of standard size flumes that do not conform with the specified dimensions of throat width or upstream measuring distance.

A further investigation is needed to examine the effect of varying the side angle or bottom angle but by use of this non-dimensional method, the effect of these variables could be found by experiments on perhaps not more than three sizes of flume.

ACKNOWLEDGMENTS

The writer wishes to thank Peter L. Monkmeyer for his useful suggestions.

APPENDIX.—NOTATION

The following symbols have been adopted for use in this paper:

- b, b_{cn}, b_1 = Channel widths (See Fig. 1);
- E_1 = total energy head at the measuring point x_1;
- E_o = non-dimensional energy head = $\frac{E_1}{b}$;
- f_1, f_2, and so forth = arbitrary functions;
- g = acc^n due to gravity;
- K = discharge coefficient for parallel flow at throat;
- K_e = as defined in formula $Q = K_e\, b g^{1/2}\, y_1^n$;
- K_p = discharge coefficient for curved flow at throat;
- n = as defined in formula $Q = K_e\, b g^{1/2}\, y_1^n$;
- Q = discharge;

Q_o = non-dimensional discharge $\frac{Q}{g^{1/2} b^{5/2}}$;

V_1 = velocity at measuring section;

V_c = true critical velocity at throat;

x_1 = distance from throat crest to measuring section (See Fig. 1);

x_o = non-dimensional distance = $\frac{x_1}{b}$;

y_1 = depth at measuring section;

y_o = non-dimensional depth = $\frac{y_1}{b}$;

y_c = true critical depth at throat;

y_{cn} = nominal critical depth given by $y_{cn} = \sqrt[3]{\frac{Q^2}{b_{cn}^2 g}}$;

λ_1, λ_2 = factors correcting for non-hydrostatic pressure, friction and non-uniform velocity distribution;

β = width ratio = $\frac{b}{b_1}$;

ϕ = bottom angle (See Fig. 1);

θ = side wall angle (See Fig. 1).

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FRIANT - SAN JOAQUIN RIVER LITIGATION

By Charles H. Lee,[1] F. ASCE

SYNOPSIS

The Friant - San Joaquin River litigation in the Federal courts, popularly known as Rank versus Krug, involved a wide scope of current issues in the field of water law in the Western United States, as well as court recognition of unusual geologic and hydrologic features of a stream system and its related ground water. It is also interesting as an example of water conservation to be accomplished through an engineer's "physical solution" incorporated in a court order. The paper states briefly the issues, the history of the case, and highlights of the decision with striking extracts from the opinions of the district and circuit courts.

The physical solution consisted of a series of ten check dams with collapsible gates to be constructed in a 50-mile reach of San Joaquin River below Friant Dam. It created a series of ponds whose aggregate bank and bed area, and depth of water continuing throughout the year, would simulate that of the natural flow of the stream during the melting snow period April 1 to September 30.

The court recognized seepage from the river as the source in part or whole of ground water in wells over an area of approximately 300,000 acres comprising the alluvial cone of San Joaquin River.

Note.—Discussion open until May 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR 4, December, 1961.

[1] Senior Partner Lee and Praszker, Cons. Engrs., San Francisco, Calif.

The complaint was brought as a class action by fourteen individual riparian and overlying landowners in behalf of all landowners similarly situated. Plaintiffs also included the city of Fresno and the Tranquillity Irrigation District.

INTRODUCTION

The Friant - San Joaquin River litigation[2] is of particular interest to civil engineers because it highlights the impact of large scale water development on the ordered system of property rights in the use of water that has been built up in the western United States during more than 100 yr (1850-1960) of irrigation practice. It also illustrates the manner in which engineers can work with the courts in devising economically feasible "physical solutions" that will facilitate complete use of the fluctuating flow of a natural stream, without encroachment on the property rights of existing water users. An unusually interesting subsurface water problem in applied geology and hydrology is also involved.

The litigation arose when the United States Bureau of Reclamation Department of Interior (USBR) built the Friant Dam on San Joaquin River as part of the Central Valley Project of California and in 1944 started to regulate and divert the flow for irrigation of distant lands through the Madera and Friant-Kern Canals (Fig. 1). This diversion was attempted by virtue of possessing the structures capable of physical control of the whole flow of the river but without complete acquisition of water rights of downstream users.

The plaintiffs comprise fourteen individual riparian and overlying land owners, one private corporation, the city of Fresno, and the Tranquillity Irrigation District. Plaintiff's lands and water right holdings lay along the San Joaquin River and on the alluvial cone of the river below Friant for a distance of 59 miles to Mendota Pool. These parties sued, "not only on behalf of themselves, but as representatives and members of a class owning rights to the use of water from a common source of supply, vis: The San Joaquin River as it flowed between Friant and Mendota prior to Friant Project." Two methods of taking water were practiced. Riparian land owners pumped directly from the river. Water for more distant portions of lands bordering the stream and for lands not bordering the stream was obtained by pumping from wells found by the court to be supplied in whole or in part by water seeping and flowing underground from the San Joaquin River. The trial court found that lands of the class of landowners represented by plaintiffs comprised approximately 300,000 acres overlying the alluvial cone of San Joaquin River westerly of Friant, the exterior boundaries of which were defined by the so-called "Lee Lines" (Figs. 1 and 2). These lands had been highly cultivated for many years with ample irrigation water supply derived both from San Joaquin River and by gravity ditch water from the Fresno Irrigation District which derived its supply from Kings River, an adjacent stream to the south. The lands included a portion of the city of Fresno, whose municipal supply was obtained from wells supplied in part by subsurface flow from the San Joaquin River, and in part from return water from irrigation within the Fresno Irrigation District.

[2] Rank et al., versus (Krug) United States of America et al., Southern Dist. of California, Northern Div., No. 685-ND Trial Court decree entered June 20, 1957.

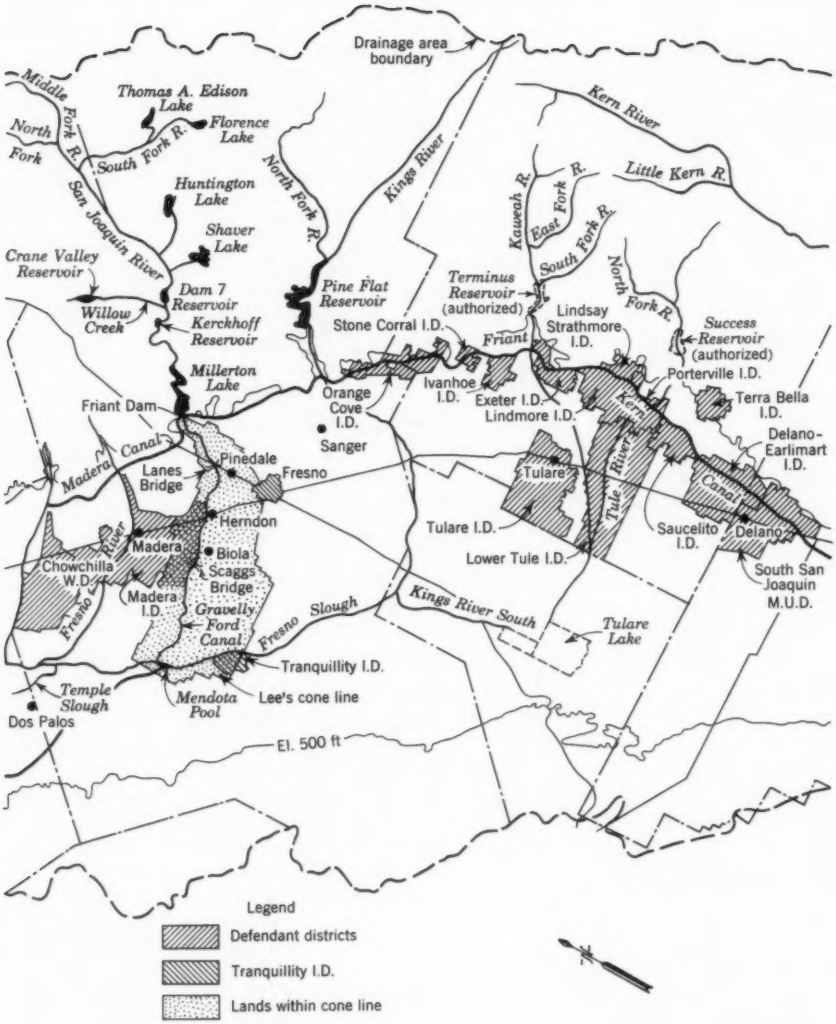


FIG. 1.—PHYSICAL FEATURES OF THE SAN JOAQUIN RIVER WATERSHED

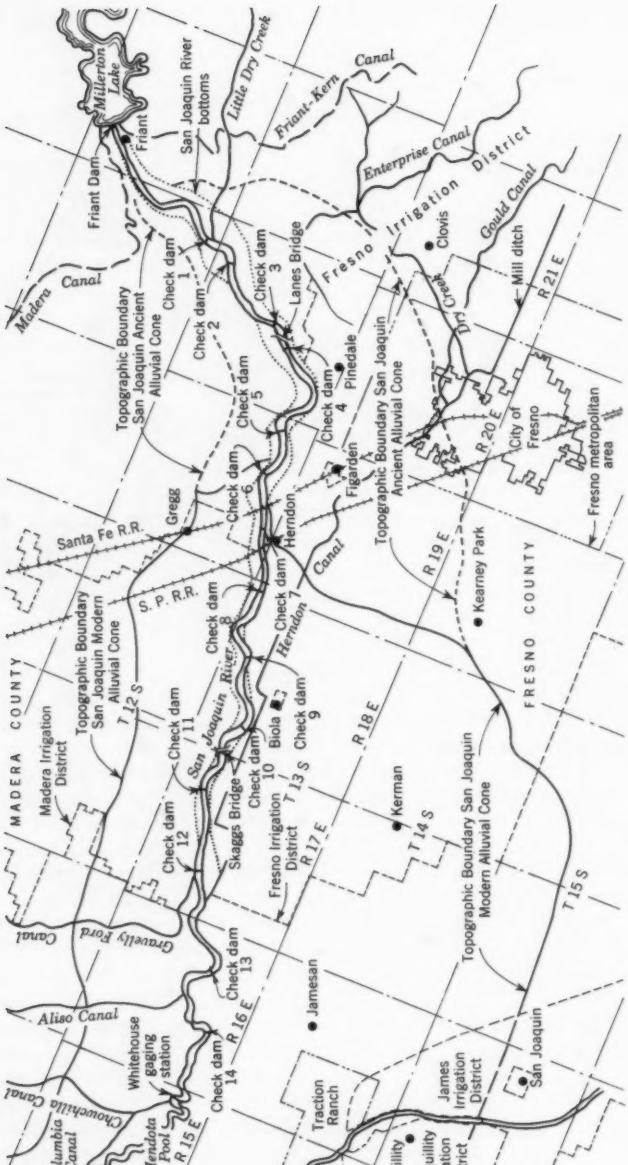


FIG. 2.—OUTLINE MAP OF SAN JOAQUIN RIVER BOTTOMS AND LANDS NEIGHBORING THE RIVER IN MADERA AND FRESNO COUNTIES

Defendants were the Secretary of Interior of the United States, certain employees of the USBR, and fifteen Irrigation districts that had contracted to take water from Friant-Kern Canal. The State of California appeared as Intervenor. During the course of the trial, the United States of America was named defendant.

The plaintiffs sought to enjoin the acts of defendants that they claimed were contrary to established law. They did not seek to prevent the construction or operation of the Central Valley Plan or portion thereof, but maintained that the lawful operation of the Plan requires recognition and enforcement of their rights to water in accord with applicable Federal and State laws.

Quoting from the decision of the trial court;

> "While the prayer of plaintiff's complaint asked the full flow of the river, their counsel has insisted throughout the trial, and the case was tried on the basis, that the plaintiffs seek only that reasonable amount of water which they insist they are entitled to have for reasonable present and prospective uses by reasonable methods of diversion under the applicable Federal and California laws in the form of a judgment requiring a "physical solution." In relation to the full flow of the river and the amount impounded and diverted at Friant, the amount thus sought is small."

> "The real difference as it has finally developed is the claim that the United States has taken the entire flow of the San Joaquin river by the exercise of its power of eminent domain, and that whatever rights the plaintiffs have to water to be released from Friant is solely in the discretion and determination of government officials. The question is, therefore, whether the plaintiffs are entitled to uncertain releases as a matter of mere grace in the amounts determined from time to time by government officials, or to releases of water as a matter of adjudicated right by virtue of their status as downstream and overlying owners with vested rights under applicable Federal statutes and the California law."

The suit was first filed in a state Superior Court, September 25, 1947 but was transferred to the Federal District Court October 6, 1947. Active trial commenced July, 1951 covering a period of 210 court days. The transcript comprised 30,000 pages, and over 800 exhibits were admitted in evidence, most of them of a technical nature. More than 70 witnesses gave testimony; that of individual engineers extended over weeks or months. The principal witness for plaintiffs was under cross-examination for 11 weeks. The trial court found from the evidence in the case that all of the lands lying within the "Lee lines" have a common source of supply—namely, the waters of San Joaquin River as it flowed before construction of Friant Dam and the Madera and Friant-Kern Canals.

The court found, further, that the rights of plaintiffs and their class as riparian and overlying landowners are paramount and superior to all claims of right by the United States and all other defendants, and that only that portion of the flow of San Joaquin River at Friant which is not necessary to satisfy those rights is surplus water and subject to appropriation under the terms of California constitution and laws.

The court also found that unless, in the exercise of its equity powers, injunctive relief was not given to protect the rights of plaintiffs and their class from illegal diversions, that the damage would be great, immediate and irreparable and would be continuing from year to year.

The court found that, of the various plans submitted by litigants, the plaintiffs plan of physical solution would most nearly meet the requirements of the California State Constitution for conservation of water by reasonable and beneficial uses, that it would prevent waste, and would guarantee the protection of the prior-vested rights of plaintiffs and their class. The court ordered that the plaintiffs plan, to the extent of ten check dams, be put into effect by the USBR but retained jurisdiction for the purpose of enforcing the judgment and decree, and after construction and operation of the plan of physical solution to modify or change the same as experience may show to be necessary to protect the rights and duties of the respective parties.

The decision of the United States Court of Appeals, filed March 31, 1961, affirmed the judgment of the District Court, with two exceptions; first, the joinder of the United States of America as a defendant, and, second, the terms on which the city of Fresno is entitled to receive water from the United States at Friant Dam. With regard to condemnation, the Court of Appeals stated:

"In an exercise of its power of eminent domain, then, the United States must commit itself as to what is taken and as to what remains untaken. That which remains untaken and continues vested in the owner, the officers of the United States must continue to respect."

"In the case at bar, the operation of Friant Dam was not of such a character as to notify these plaintiffs as to the extent of the seizure of their rights. Nor was it accompanied by any sufficiently definite uttered or written notification."

"We conclude that the water rights of these plaintiffs have not been acquired by the United States through exercise of its power of eminent domain."

In comment, the Court stated:

"The decree of physical solution is not then a detriment imposed upon the Bureau. It is a grant of right to the Bureau and a detriment or limitation upon the rights of the plaintiffs to the full natural flow of the river. The judgment of the Bureau to accept the physical solution or, in the alternative, to reject it and resort to condemnation remains available."

"The question is as to the extent of freedom which Bureau should enjoy in operating Friant Dam for project purposes. We are concerned here with a balancing of the public interest which this project represents and the private interests represented by these plaintiffs. We cannot say that the district court was in error in determining the balance to fall in favor of the plaintiffs."

"The question is whether these plaintiffs should be required to live with such free governmental control. A water right with no assurance of its peaceful enjoyment is worth little to a farmer whose very existence is dependent upon the predictability of his future water supply. For this reason, the injunction is one of the traditional forms taken for the judicial establishment of water rights."

PHYSICAL SOLUTION

The physical solution proposed by plaintiff, and accepted by the court, was essentially a plan by which the natural flow of the river with its seasonal varia-

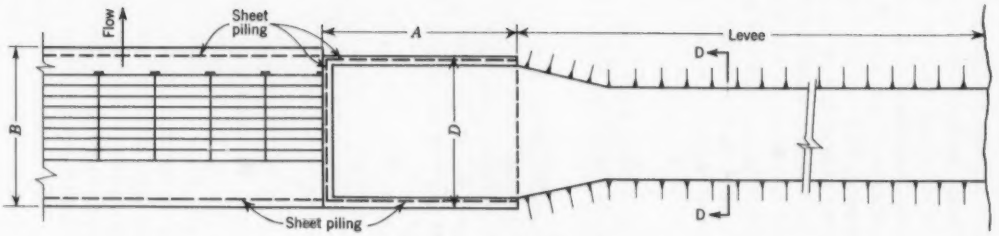
tions could be simulated. The average annual natural flow of San Joaquin River at Friant is approximately 2,500 sec ft, with monthly averages during the snow-melting season May to July of 10,000 sec ft to 25,000 sec ft. Minimum flow occurs October to December, averaging 250 sec ft to 500 sec ft. There is a resulting wide variation in channel width and height of bank, as well as velocity and head, all of which directly influence channel loss by seepage and percolation. Tests and studies submitted in testimony by plaintiff indicated that, percentage-wise, the rate of seepage loss from the San Joaquin River increased with and was at a more rapid rate than the rate of discharge. At annual average discharge rate, seepage amounted to 150 sec ft, or at the rate of 2.5 sec ft per mile of river channel. Testimony by a defendant engineering witness, with local experience, was that percolation between Friant and Mendota exceeded 200,000 acre-ft, per yr. Such water constitutes the natural source of well waters within the alluvial cone of San Joaquin River. The proposed physical solution was designed for the purpose of perpetuating this supply with concurrent diversion of water at Friant Dam, as well as providing an accessible supply for direct riparian pumping.

Plaintiffs contended that any physical solution to successfully meet the situation created by diversion of water at Friant Dam by the USBR should accomplish the following:

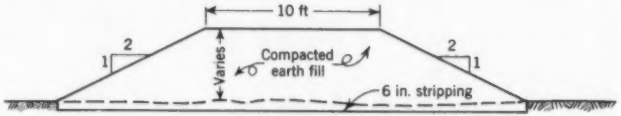
1. Maintain a water surface elevation in San Joaquin River at all river pumps between Friant and Gravelly Ford equal to the prevailing level prior to storage and diversion at Friant Dam, or at least high enough to maintain good operating conditions without sucking air or taking sand and without extension of suction pipes more than 10 ft beyond what they were prior to filing of this action.
2. Maintain a wetted perimeter, water depth, and permeability on and in the bed and banks of San Joaquin River between Friant and Mendota Pool, equivalent to that prevailing during the period April 1 to September 30 of a normal runoff year prior to storage at Friant Dam.

The plaintiffs' plan for accomplishing these purposes was the construction and operation of a series of fourteen check dams with collapsible gates (Fig. 2). The judge in his decision reduced the number to 10. These dams were designed to create a continuous series of ponds extending full-length of the river channel from Friant to Mendota Pool, varying in water depth from 4 ft to 11 ft, whose average water level would be roughly equivalent to that of the average annual discharge of the river. It was recognized that this would not provide a seepage quantity commensurate with that during the melting snow period but, to compensate, it was proposed to maintain the ponds throughout the year. Collapsible dams were provided so that they could be lowered at times of flood release from Friant Dam and for 24-hr scouring releases of 4,000 sec-ft during April, June, and August. Testimony indicated that the amount of water required to operate these ponds, including evaporation, transpiration, channel seepage, riparian pumping, and scouring releases would be 183,000 acre-ft annually, or about 10% of the average annual flow of the river at Friant. The proposed flow criterion for the rate of release of water at Friant Dam at any time was that quantity which would maintain a flow of 10 sec-ft over the lowest dam on the river. This would insure that every pond was full and, in conjunction with periodic scouring releases, that maximum seepage rates were maintained.

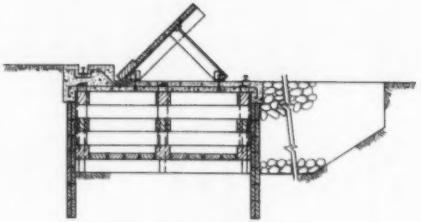
The proposed check dams consisted of a reinforced concrete deck 12 ft to 20 ft wide extending full width of the stream at stream bed elevation with



PARTIAL PLAN, TYPES II AND III



SECTION D-D



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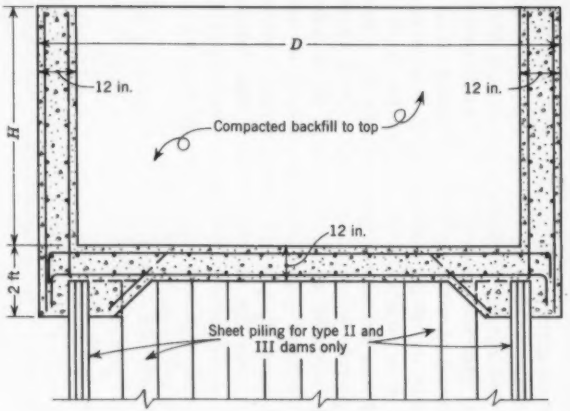

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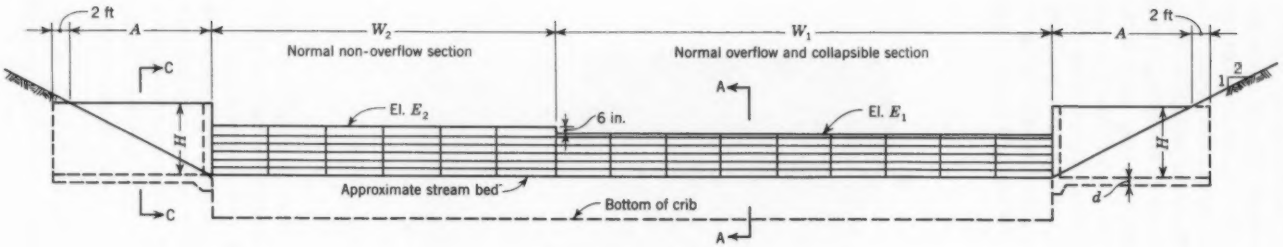
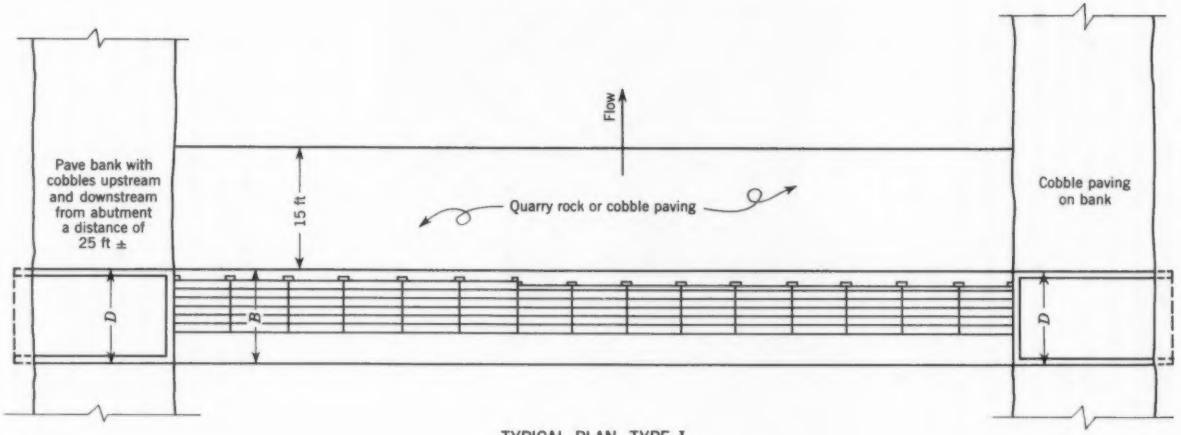


TYPICAL UPSTREAM ELEVATION



TYPICAL PLAN, TYPE I

FIG. 3.—TYPICAL COLLAPSIBLE CHECK DAMS

concrete abutments consisting of headwall, wing walls, and floor tied into the banks with heavy rock bank paving upstream and downstream (Fig. 3). Foundations were of three types to conform with subgrade stream bed formations. Type I foundation was timber crib backfilled with large cobbles adapted to stream bed formation of cobbles and coarse gravel to considerable depth. Type II was a floating foundation consisting of two lines of Wakefield sheet piling, driven to 10 ft depth along the upstream and downstream edge of the deck, the tops of which were incorporated within the deck. This type was adapted to the middle and lower portions of the river where formation was sand or silt to considerable depth. Type III was adapted to sand or silt stream bed with firm hardpan or clay within 10 ft to 20 ft depth, and consisted of 12 in. wooden piles along the downstream edge of the deck and Wakefield sheet piling along the upstream edge.

FIG. 4.—GANTRY CRANE OPERATING ON CHECK DAM

The concrete deck supported a series of collapsible wooden gates, a steel radial gate, and two lines of rails for wheel-supported gantry crane operation. The gates were 6 ft to 8 ft in length and 4 ft to 12 ft in width depending on the depth of water. The gates were hinged to the deck and supported by steel struts when in raised position. It was planned to raise and lower gates from a gantry crane equipped with a gasoline motor, and an auxilliary motor for propulsion. Similar dams have been constructed on several California streams. Fig. 4 shows a gantry crane in operation on such a dam across the Feather River near Oroville.

Rank versus Krug, by providing a physical solution, points the way toward greater utilization of the natural ground water reservoirs of the San Joaquin

and Sacramento Valleys. The case has a warning that should be heeded by landowners and communities on every major stream cone where comprehensive surface storage and diversion systems are proposed.

Friant Dam was built only to moderate height and creates storage wholly inadequate for cyclic storage of stream runoff, or even complete monthly storage regulation. Engineering design of the Central Valley Project was based on extensive use of subsurface storage at great distances from the San Joaquin River alluvial cone, for which purpose large canal capacity was provided for maximum diversion of winter and spring runoff. The plan was to spread such water over the irrigable lands of the valley where it was assumed that it would be absorbed, filling the underground storage basins and, thus, providing a supply that could be pumped from wells during periods of shortage. Consideration of soil types prevalent in the valley, however, shows that much of the irrigable land under the canals is relatively impermeable and underlain by hardpan. In addition, there is the human element of persuading farmers to buy water in the winter time to irrigate their lands when they are occupied by other urgent farm operations. Water logging or leaching of the soil would also present problems. Seepage from the channels of the main streams tributory to the valley was the original natural source of replenishment of subsurface water. It would appear to be better engineering design to fully utilize these natural channels rather than abandoning them to undertake untried artificial methods of ground water replenishment in other areas. The physical solution ordered by the court on San Joaquin River is in furtherance of the idea of full utilization of the absorptive capacity of natural channels.

The estimated cost of constructing the ten check dams ordered by the court is approximately $1,000,000. This is a small fraction of the probable cost of condemning the necessary lands and water rights.

SUBSURFACE WATER MOVEMENT

A basic issue underlying the decision of the trial court was the structure of the alluvial valley floor deposits beneath the San Joaquin River and adjacent lands within the "Lee lines," which at some points extend out from the river to a distance of 10 miles, and the relation of this structure to the subsurface distribution of water seeping from the channel of the river.

The contention of the government was that these deposits were homogeneous, sieve-or sponge-like, that return from gravity irrigation water applied on the surface could readily percolate downward to the main subsurface water body, but that water seeping from the river was confined to the immediate vicinity of the river channel because of the higher elevation of the surface of ground water as maintained by return from irrigation in the Fresno Irrigation District.

The plaintiff on the other hand presented geological and engineering testimony indicating that these deposits had a complex and heterogeneous structure and that, geologically, they comprised the alluvial cone of San Joaquin River. Alluvial cones are a recognized geological structure typical of semi-arid and arid regions. They are composed of alluvial debris gathered by a stream from its mountain watershed, transported to the adjacent structural trough and there deposited opposite the mouth of the mountain canyon in the form of a cone, often flattening out at its margin to form a fan. Such cones are composed of boulders, cobbles, gravel, sand, silt, and clay. The coarser materials are deposited first along stream channels radiating out from the mouth of the canyon and extending well into the valley. At lower levels, as the water loses velocity in flowing

out onto the valley floor, the finer materials are deposited. The location of stream channels shifts from time to time as they fill up with coarse debris and, finally, the channels become buried with finer material deposited either from sheet flow or by wind action. The buried channels of coarse material are permeable and transmit subsurface water as percolating flow more easily than do the finer materials that encase them. The channels of coarse material also readily transmit hydraulic pressure. Those systems of channels radiating out from the canyon mouth at different levels are natural feeders that penetrate throughout the mass of the alluvial cone, conveying percolating water and transmitting hydraulic pressure just as does the arterial system that radiates out from the human heart. Geologists term these underground channels "aquifers" from the Latin words meaning "water carriers." Much of the voluminous technical testimony in the case dealt with the geology and hydrology of the San Joaquin River Alluvial cone; first, as to its existence and the existence of aquifers and, second, as to whether or not the cone is fed by seepage from the flow of San Joaquin River and whether this seepage is an appreciable element in the supply of wells extracting water from the cone. On all of these questions, the findings of the trial judge were in the affirmative.

A profile following lines of southerly projecting aquifers from the head of San Joaquin River Cone to the City of Fresno is shown in Fig 5. This diagram is based on exhibits and testimony in the case of Rank versus Krug and on Exhibit No. 70 submitted by the city of Fresno at the subsequent hearing before the California State Water Rights Board on the applications of the city of Fresno, the Fresno Irrigation District, and the USBR to divert unappropriated water from the San Joaquin River. This profile extends beyond the alluvial cone of San Joaquin River to include that of the several small creeks and, finally, that of Kings River. It is probable, as indicated by a government geological witness, that at depth the alluvial cone of San Joaquin River extends southerly beneath and interbedded with alluvial cone material deposited by the small creeks and is finally terminated by the alluvial cone of Kings River. The natural level of ground water in wells as fed by the river prior to surface irrigation from Kings River is shown by the water level of 1870 derived from shallow well data in early government reports. The water levels in shallow wells after 25 yr of irrigation are indicated by the line of 1900. The effect of irrigation (for location of canals, see profile and Fig. 2) was to build up a ridge of semi-perched ground water between the river and Fresno, supported both by impervious beds within the alluvial cone and by the upward pressure of ground water from the river at the 1870 level. This ridge, although progressively lowering with extension of pumping from wells, has persisted through the years, as indicated by the lines of 1954 and 1956, prepared from static water level contour maps made annually by the State of California. These measurements were made on both shallow and deep wells in the fall of the year after the active pumping season. Measurements of standing water level made by the writer in 1958 indicated that at that date the ground water ridge had practically disappeared.

Supplemental measurements made at this time indicated that pumping drawdown in the various wells was from 13 ft to 20 ft. Pumped wells, as shown on the profile, are practically all open-bottom, penetrating coarse water-bearing sand and gravel lying immediately below beds of impervious clay and sandy clay. Water level observations made simultaneously on pairs of adjacent shallow and deep wells widely distributed over the area did not show any effect in shallow wells when pumping was started or stopped in deep wells; this confirmed the fact of the relative imperviousness of the clay beds. The immediate

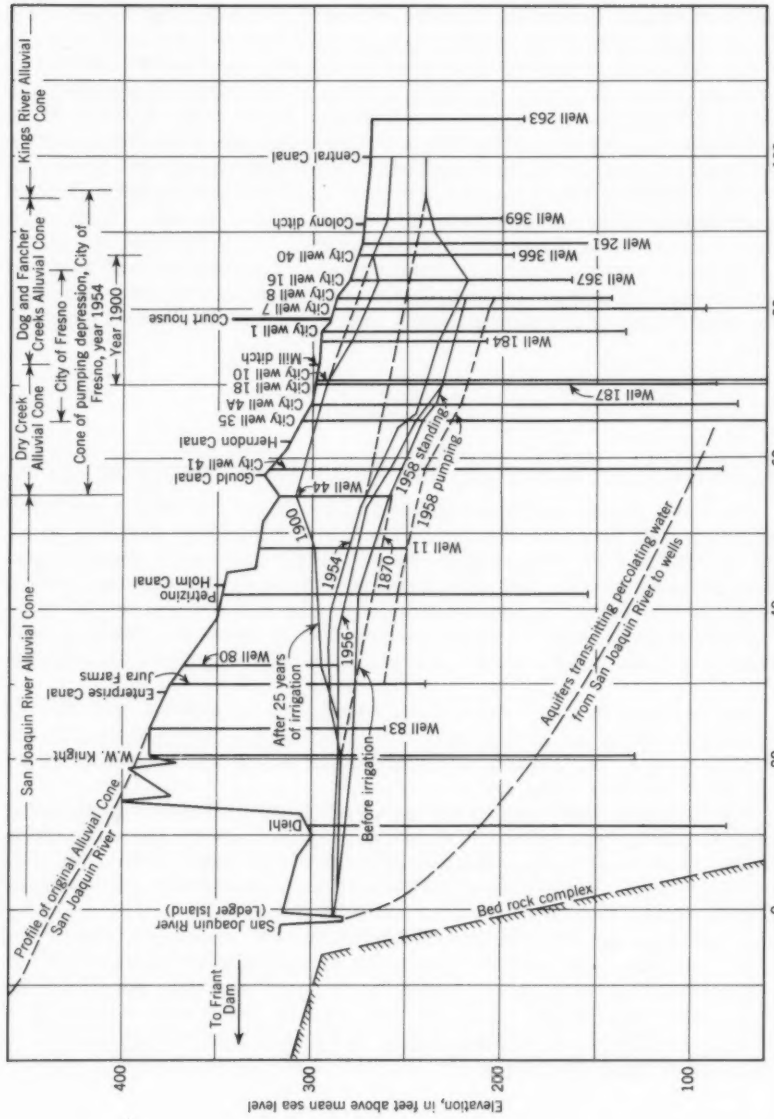


FIG. 5.—PROFILE OF GROUND WATER LEVEL IN WELLS

source of supply to pumped wells thus appeared to be water percolating freely through the deep lying permeable gravel aquifers fed directly from the river channels, as indicated by Fig. 5. This source is hydraulically available to wells in the city of Fresno, as shown by the fact that pumping levels even as early as 1900 were many feet lower than the level of water in San Joaquin River at the head of the aquifers. In 1958, pumping level in City Well No. 8 was 78 ft lower than the river level, representing an overall hydraulic slope of 5 ft per mile.

Continued pumping from municipal and irrigation wells since 1944 has created an extensive area of pressure depletion in the lower water-bearing strata within and to the north of the city of Fresno. This has resulted in lowering the hydraulic support beneath the body of semi-perched ground water derived from surface irrigation, thus inducing a slow vertical leakage through the impervious clays resulting in general lowering of water levels in wells. The source of pumped water has thus been twofold, partly from the river and partly from overlying irrigation. As urbanization extends into irrigated areas east and north of Fresno, the proportion of water derived from irrigation will decrease, and the importance of the natural supply from the river will correspondingly increase.

CONCLUSIONS

Conclusions to be drawn from this litigation may be stated as follows:

1. The statutory and legal system of property rights in the use of water that has been built up in the western United States from 1850 to 1960 is firmly established and recognized, both by land owners and the state and federal courts. This status exists notwithstanding efforts to overthrow it under the legal theory of paramount Federal stream system rights, or advocacy of the practical engineering advantages for large water development projects of consumer distribution on a public utility basis at the pleasure and convenience of management.

2. The decision in Rank versus Krug recognized the alluvial cone and fan of San Joaquin River as a hydrologic unit including the places of natural and artificial recharge, the places of artificial discharge, and the intervening rocks through which the water may move.[3]

3. The decision also recognized the physical occurrence of percolating subsurface flow through relatively deep-lying aquifers where hydraulic heads and slopes are favorable, even though superimposed by a zone of semi-perched ground water whose water table is at elevations higher than the source of the percolating water.

4. A "physical solution," dependent for its success on correct engineering interpretation of geologic and hydrologic facts, as well as workable construction and operational features, was accepted by the court and incorporated in the decision with requirement that it be executed by defendants in the case in order to satisfy the water rights of plaintiffs and permit of storage and diversion of surplus waters without waste or invasion of existing rights.

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[3] Compare with ASCE Manual of Engineering Practice No. 40, Ground Water Basin Management p. 27, 1st par.

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DESIGN OF INTERCEPTOR DRAINS IN HETEROGENEOUS SOILS

By R. William Nelson[1]

SYNOPSIS

The design of interceptor drainage systems is complex and often hampered by lack of methods for analyzing variable permeability. An approximate method for including variable permeability in a design procedure is presented.

The analysis requires a watertable contour map for transposing the flow system in heterogeneous soil, to an equivalent system, having constant permeability throughout. In the uniform system equations derived for homogeneous soil can be applied. Reverse-transformation of the prediction from the uniform soil system to the field problem gives the expected condition after drainage facilities are installed.

INTRODUCTION

Rational design lies at the base of the great advances experienced in engineering disciplines during the past century. This progress was accomplished through understanding and describing, often through mathematics, the physical phenomenon involved. Similarly in drainage design analytical methods should be expected to provide the needed tools. However, the drainage designer more often than not finds himself relying almost exclusively on intuition or "rules of thumb."

Note.—Discussion open until May 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR4, December, 1961.

[1] Engr., Geochemical and Geophysical Research, Hanford Atomic Products Operation, General Electric Co., Richland, Wash.; formerly, Agric. Engr., Western Soil and Water Mgt. Research Branch, Agric. Research Service, U. S. Dept. of Agric., Fort Collins, Colo.

Why then has rational design in drainage developed slower than in other disciplines? Contributing factors are: (1) biological and climatic conditions make the criteria defining adequate drainage difficult to establish; (2) the soils in which drains are installed are heterogeneous and accordingly are difficult to characterize; and (3) the consequence of malfunction is not always obvious and is often less severe than in other engineering disciplines.

In recent years, studies were started which probably will eventually produce criteria defining adequate drainage. The heterogeneity of soil permeability and its effect on drainage design has received and is receiving far less study. In fact, heterogeneity has rarely been analyzed but rather tolerated through statistics. Unfortunately such statistical approaches have usually measured deviations from uniformity rather than considered heterogeneity as the characteristic tendency of soil. However, progress in considering nonhomogeneous soils is needed because the gulf between theoretical and practical applications is often the difference between homogeneous and heterogeneous soils.

An approximate method for treating heterogeneity in interceptor drainage design is presented herein. Heterogeneity of permeability is treated through transforming the field problem into an equivalent system of uniform permeability. Solutions assuming a homogeneous media can than be applied to predict conditions after drainage. The predictions are extended to the heterogeneous field problem through a reverse transformation.

THEORETICAL BASIS

Two earlier papers provide a theoretical basis for measuring heterogeneous permeability as well as justifying the approximation to be used in the transformation. In the first paper[2] two facts pertinent to this discussion were established. They are:

1. The saturated permeability distribution in any heterogeneous system can be determined by studying the steady piezometric head distribution along streamtubes.
2. The permeability distributions along parallel streamtubes are independent of each other insofar as the piezometric head or potential is concerned. Hence, in addition to item 1, the boundary condition must specify the permeability along some equipotential (it cannot be along a streamline) which completely traverses the region of interest.

The second paper,[3] which described an experimental examination in two dimensions of the theory of the first paper,[2] provides the following:

3. An error function was obtained for the linearized flow approximations along steamlines in lateral flow systems. In general this error is less than 5% at critical points in the region of analysis with an average regional error of about 1%.

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2 "In-place Measurement of Permeability in Heterogeneous Media I. Theory of a Proposed Method," by R. William Nelson, Journal of Geophysical Research, Vol. 65, No. 6, June, 1960.

3 "In-Place Measurement of Permeability in Heterogeneous Media II. Experimental and Computational Consideration," by R. William Nelson, Journal of Geophysical Research, Vol. 66, No. 7, July, 1961.

4. The combined linearization error and unavoidable graphical error resulting from construction of the flow net and scaling of distances result in an average error of 1% to 2.5%.

These four factors provide the basis for the transformation method to be described here. Items 1 and 2 show that the variation of permeability is uniquely determined by the piezometric head along the streamtube. In essentially lateral flow systems the water table elevation is a good approximation of the piezometric head so the relative permeability can be found directly from the water table contour map. The error for this approximation (item 3) provides a way to examine any questionable case and determine the maximum linearization error at critical points. Item 4 shows the combined linearization and construction error to be small. Accordingly the water table shape along a streamtube will be used to transform the field problem of heterogeneous soil to a hypothetical equivalent system of uniform permeability.

QUALITATIVE CONSIDERATION OF TRANSFORMATION

A qualitative idea of the transformation, will be examined through a hypothetical example before verifying the transformation or applying the method to an actual field problem. A tilted flume filled with soil of uniform permeability through which water is flowing steadily is shown in Fig. 1. The water table is a straight line between El. 10 and El. 8 and the hydraulic conductivity is 1 in. per hr.

The second tilted flume (Fig. 2) contains the same soil except for a column of less pervious material midway between El. 10 and El 8. In the second flume the water table shape (solid line) has changed from a single straight line as in Fig. 1 to three-line segments in Fig. 2.

If a model of the flow system shown in Fig. 1 is desired using a sand with a different hydraulic conductivity (K_t) of 1.33 in. per hr three things are required: (1) The elevation at A and B must be 10 ft and 8 ft respectively, (2) H must be the same or very nearly so, and (3) the discharge must be identical. The hydraulic conductivity, however, is 1.33 times larger than the original value of 1 in. per hr in Fig. 1. To compensate for the permeability change, let the flume length be increased by a factor of 1.33. In effect, such a compensation has been accomplished by making the ratios of permeability and length equal, i.e., $K_O/L_O = K_t/L_t$.

Using this same scheme let a third flume be built modeling the one shown in Fig. 2 but of sands of different permeability. Again use sand with K_t = 1.33 in. per hr. The first 49.5 ft downstream from point A in Fig. 2 must be lengthened by a factor of 1.33/1 or increased to a length of 65.8 ft. The column of less pervious material (K = 0.1) of 1-ft length is represented by 1 times 1.33/0.1 or 13.3 ft of the material having a hydraulic conductivity of 1.33 in. per hr. The last 49.5 ft of K= 1.0, just as for the first 49.5 ft, is lengthened to 65.8 ft. Therefore, the total flume length in Fig. 3 will be 144.9 ft. The material in the model now has a hydraulic conductivity of 1.33 in. per hr throughout the flume length, and since it is uniform, the water table will be a straight line from A to B.

If the valves of hydraulic conductivity are not known, a model of Fig. 2 may be constructed by drawing a dashed straight line connecting points A and B.

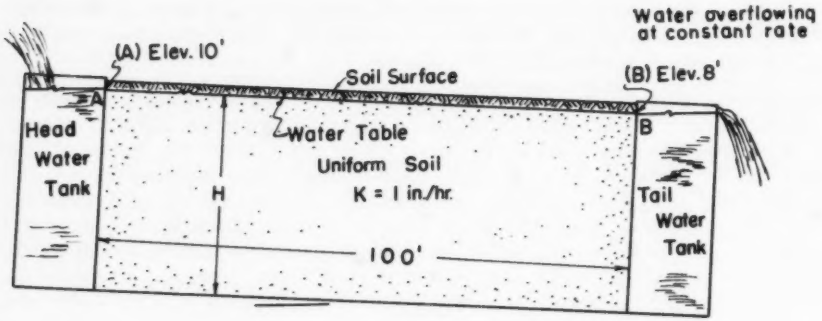

FIG. 1.—TILTED FLUME WITH WATER FLOWING THROUGH UNIFORM SOIL

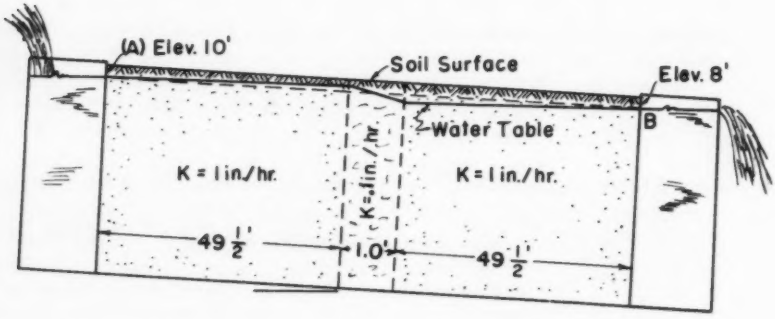

FIG. 2.—WATER FLOWING THROUGH A TILTED FLUME CONTAINING SOIL OF DIFFERENT HYDRAULIC CONDUCTIVITY

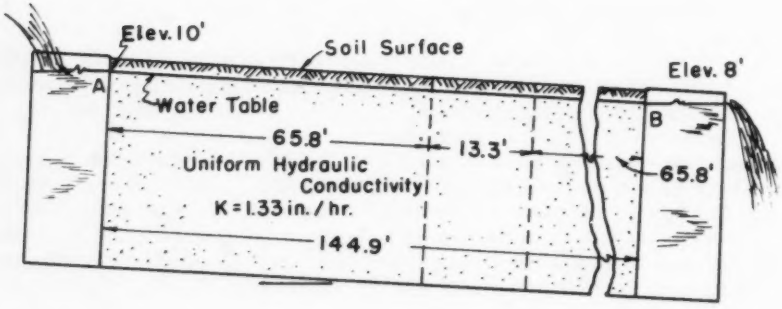

FIG. 3.—FLUME CONSTRUCTED AS A MODEL OF FIG. 2 WITH UNIFORM CONDUCTIVITY HYDRAULIC

The transformation is accomplished by shortening or compressing the first 49.5 ft until the original water table coincides with the dashed straight line, meanwhile lengthening the less pervious column and then again shortening the second 49.5 ft until the resultant total length equals 100 ft. When this is accomplished the original irregular water table (solid line, Fig. 2) will coincide with the dashed straight line between A and B.

Once an equivalent uniform system has been found drainage equations for uniform soils can be used to predict the drawdown curve. The drawdown curve in the equivalent uniform system can then be transferred back to the nonuniform problem. The validity of this transformation will be demonstrated in the following section.

VERIFICATION OF TRANSFORMATION

Considering one-dimensional flow along a streamtube and allowing the area available for flow to vary within moderate limits, then combining Darcy's law with linear continuity gives:

In the heterogeneous system,

$$Q_v = K_v H_v W_v \frac{d\phi}{ds} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

and in the equivalent uniform system,

$$Q_u = K_u H_u W_u M_u \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which Q_v and Q_u are the discharges or volume rates of flow in the heterogeneous and uniform systems respectively, K_v and K_u are the (hydraulic) conductivities or permeabilities in the respective systems, H_v and H_u are the respective flow depths, W_v and W_u are the respective flow widths, $M_u = \frac{d\phi}{du}$ is the water table slope in the equivalent uniform system, and $\frac{d\phi}{ds}$ is the potential gradient along the streamline in the variable system.

In order to have an equivalent uniform system Q_v must equal Q_u, therefore

$$K_u H_u M_u W_u = K_v H_v W_v \frac{d\phi}{ds} \quad \ldots\ldots\ldots\ldots \quad (3a)$$

or

$$M_u = \frac{K_v}{K_u} \frac{H_v}{H_u} \frac{W_v}{W_u} \frac{d\phi}{ds} \quad \ldots\ldots\ldots\ldots\ldots \quad (3b)$$

Select any convenient point P' in the equivalent uniform system having a distance coordinate $u = 0$. Then associated with P' let there be a point P in the heterogeneous system having a distance coordinate $s = 0$. If the actual and hypothetical uniform systems are to be equivalent at points P and P' then $K_u = K_v$, $H_u = H_v$, and $W_u = W_v$. Upon substituting into Eq. 3b,

$$M_u = \left(\frac{d\phi}{ds}\right)_o \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

Hence, the water table slope M_u in the equivalent system of uniform perme-

ability is equal to the water table slope, $\left(\frac{d\phi}{ds}\right)_o$, at point P in the heterogeneous system. Since $\left(\frac{d\phi}{ds}\right)_o$ is known, then M_u is also known by Eq. 4.

In the equivalent uniform system

$$M_u = \frac{d\phi}{du} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

therefore, using Eqs. 5 and 3b meanwhile dropping the v subscripts but remembering that they are variables, and replacing K_v, H_v, and W_v at P by K_o, H_o, and W_o respectively then,

$$M_u = \frac{d\phi}{du} = \frac{K\ H\ W}{K_o\ H_o\ W_o}\ \frac{d\phi}{ds} \quad \ldots\ldots\ldots\ldots \quad (6)$$

from which,

$$u = K_o\ H_o\ W_o \int_o^s \frac{1}{K\ H\ W}\ ds \quad \ldots\ldots\ldots \quad (7)$$

Eq. 7 specifies the coordinate transformation as a function of the conductance, K H W in the heterogeneous system.

In the steady heterogeneous system the quantity of flow along any streamtube is constant, therefore,

$$H\ K\ W \left(\frac{d\phi}{ds}\right) = K_o\ W_o\ H_o \left(\frac{d\phi}{ds}\right)_o \quad \ldots\ldots\ldots \quad (8)$$

or

$$\frac{K_o W_o H_o}{K\ W\ H} = \frac{\frac{d\phi}{ds}}{\left(\frac{d\phi}{ds}\right)_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

in which the terms without subscripts are at any point s along the streamtube and the zero subscripts designate the values at the point P or s = 0 (note comments between Eq. 3 b and 4). An important fact is seen by rearranging Eq. 8 slightly, that is,

$$K\ W\ H = \left(\frac{d\phi}{ds}\right)_o\ K_o\ H_o\ W_o\ \ \frac{1}{\frac{d\phi}{ds}} = \frac{\text{Constant}}{\frac{d\phi}{ds}} \quad \ldots\ldots \quad (10)$$

which indicates the water table shape is a complete result measuring not only the change in permeability but also reflects any changes in depth or width of flow within moderate limits. Accordingly, in the stricter sense, the transformation is with respect to a conductance change rather than just with respect to a permeability change. Eq. 10 justifies the use of the water table shape as the basis for transforming to the equivalent uniform system. The expression required to accomplish the transformation is obtained by substituting Eq. 9 into Eq. 7, changing the limits and integrating. This gives,

$$u = \frac{\phi - \phi_o}{M_u} \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which ϕ_o is the potential (the elevation of the water table) at point P and ϕ is the potential at any other point along the streamline in the hetergeneous

system. Expressing Eq. 11 in finite difference form,

$$\Delta u \doteq \frac{1}{M_u} \Delta \phi \quad \ldots\ldots\ldots\ldots\ldots \quad (12)$$

therefore if the change in potential, $\Delta \phi$, is zero then the incremental change in the scale of transformation, Δu, is also zero.

Although the equations just obtained are simple, their use in design would become laboriously repetitious. This undesirable feature can be overcome by using the equations in a simple graphical method for carrying out the transformation.

The conditions justifying the graphical procedure are directly obtainable from the equations just given. Eq. 10 showed the water table shape along a steamtube to be a complete measure of the change in conductance. Accordingly the whole graphical technique will be carried out on a drawing of the water table profile along a streamtube. From Eq. 4 the constant slope of the water table in the uniform system must be equal to the water table slope in the heterogeneous system at point P. Graphically this is accomplished by drawing a straight line parallel to the tangent of the irregular water table at P. Any convenient point on the water table in the heterogeneous system can be selected as P. This is possible since at any chosen point $\left(\frac{d\phi}{ds}\right)_o$, K_o, H_o, and W_o make up a unique combination of values which are related through Eq. 8 to the same four variables at every other point along the streamtube. Since any consistent set of characteristics can be assigned to the equivalent uniform system then the selection of P is arbitrary and any convenient point can be used.

The graphical mechanics of accomplishing the transformation is the last condition required for a complete graphical method. This condition is specified by Eq. 12 since a horizontal line on the water table profile represents a line along which the potential change, $\Delta \phi$, is zero. From the comments following Eq. 12, by always moving horizontally between the water tables of the two systems a one to one point correspondence of the u scale will result. A slightly different view is to realize that the potential at a point, say n, in the heterogeneous system must be identically equal to the potential at a point n' in the homogeneous system, if n' is to be the equivalent of n. Accordingly the potential difference, $\Delta \phi$, between n and n' must be zero thereby allowing only a horizontal path to be followed on the water table profile to accomplish the transformation.

The following section gives a field application of the graphical method which will further clarify and give more meaning to what has just been presented.

FIELD APPLICATION OF THE TRANSFORMATION

To illustrate the procedure in field use, consider the data taken on the Grandview, Idaho drainage project. Fig. 4 is a water table contour map for June 8, 1956. The water table contour map adequately describes the flow since from 3 to 5 piezometers each terminating at a different depth showed essentially the same peizometric head, thereby indicating primarily lateral movement. Observations on June 8, from the Grandview area were selected for two reasons: (1) This date represented approximately a steady condition based upon 1 1/2 yr of weekly measurements and (2) The approximation error at one

point for the Grandview area is the highest the writer has found to date (1961). The latter point will be discussed in detail subsequently.

Streamlines were drawn on the water table contour map as shown in Fig. 4. In actual practice each of the streamtubes making up the flow net would be considered. However, only the middle streamline of Fig. 4 will be used to illustrate the procedure. The dashed lines on each side represent the outer limits of the streamtube for which the main streamline is being considered as the average relationship. This streamtube was purposely selected because it contains the most convergence and divergence of flow of any streamtube that could be drawn on this water table contour map. Accordingly, the maximum

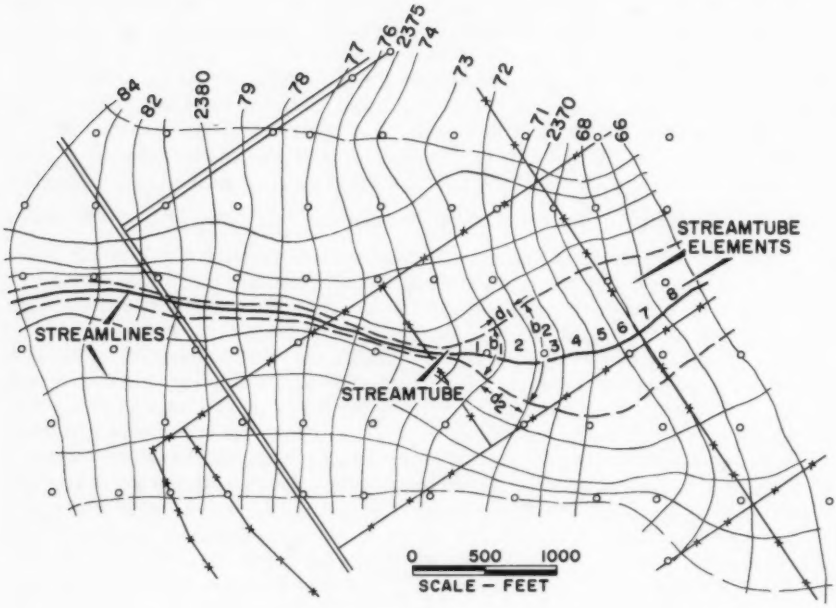

FIG. 4.—WATER TABLE CONTOUR MAP FOR JUNE 8, 1956, GRANDVIEW, IDAHO DRAINAGE STUDY

approximation error expressed in terms of the potential can be found from the relationship of an earlier paper,[3] that is,

$$\text{Percentage error} = 0.137 + 13.25 \left[\left| 1 - \left(\frac{d_1 + d_2}{b_1 + b_2} \right) \frac{1}{G} \right| \right] \quad \ldots . \quad (13)$$

in which d_1, d_2, b_1, and b_2 are as shown on streamtube element 2 in Fig. 4, and G is a geometry parameter, which can be determined experimentally and completely describes the geometrical aspects of flow in the element being analyzed.

The geometry factor, G, was determined for streamtube elements 1 to 8 (Fig. 4) through use of conductance paper. The geometrical shape of each element was transferred to a piece of "Teledeltose" conducting paper. The conductance paper was cut along the two bounding streamlines and the resistance to electrical current flow, R, measured between the two equipotentials bounding the flow net element. Knowing the specific restivity, ρ, of the conductance paper the geometry factor, G, can be found by using

$$G = \frac{R}{\rho} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

Care must be exercised to assure good contact along the two equipotentials. Otherwise the determination is easily accomplished with a standard bridge. The geometry factor, G, and the associated approximation error as computed from Eq. 13 is summarized in Table 1 for the streamtube elements shown in Fig. 4.

From Table 1 the greatest linearization error is 4.39% for element 1 which is seen to contain the greatest divergence of flow of all the flow net ele-

TABLE 1.—GEOMETRY FACTOR AND ASSOCIATED APPROXIMATION ERROR

Element designation (1)	Actual geometry factor G (2)	Geometry factor for linear approximation, G_a (3)	G_a/G (4)	Percentage error in potential (5)
1	1.433	1.083	1.7571	4.39
2	0.450	0.472	1.0471	0.76
3	0.262	0.266	1.0135	0.32
4	0.211	0.209	0.9916	0.25
5	0.199	0.241	1.2100	2.92
6	0.172	0.186	1.0824	1.23
7	0.235	0.255	1.0827	1.24
8	0.368	0.340	0.9262	1.20
			Average	1.54%

ments. It is the region of poorest approximation on the whole water table contour map and still the linear approximation error is less than 5%. The average error for the eight elements is 1.54% and the overall error for all flow net elements which could be drawn in Fig. 4 would probably be around 1%. This field example further substantiates the general statement of item 3 under the heading "Theoretical Basis." Accordingly the average overall error is less than 2.5% for the example. In general use the preceding analysis of error is unnecessary unless very drastic streamtube elements are found. Having analyzed the maximum error expected due to the linearized flow approximation, and having decided this level of approximation to be completely adequate, now return to a consideration of the graphical transformation.

A profile of the water table and gound surface along the streamline previously drawn in Fig. 4 is required. In Fig. 5 the water table profile along the streamtube is shown with Curve 1 being the water table profile. Station 20 + 40 was chosen as a convenient point to designate as P. The dashed straight line (Curve 2) in Fig. 5 was drawn parallel to the tangent to the water table at P and

at a convenient distance above the original water table. Accordingly, the condition of Eq. 4 is satisfied and Curve 2 represents the transformed water table in the equivalent system of uniform permeability. If the dashed line is considered in more detail, only the slope is important since higher or lower parallel lines serve only to slide the whole uniform system to the right or left. Accordingly a convenient distance above P was chosen to avoid confusion from too many lines during graphical construction.

Suppose it is desired to evaluate the effect of a drain placed at station 30 + 00 (Fig. 5) with the drain elevation being 70.9 ft, the location of the drain must be transformed to an equivalent location in the uniform system. This can be accomplished by moving vertically above station 30 + 00 (point D) until it inter-

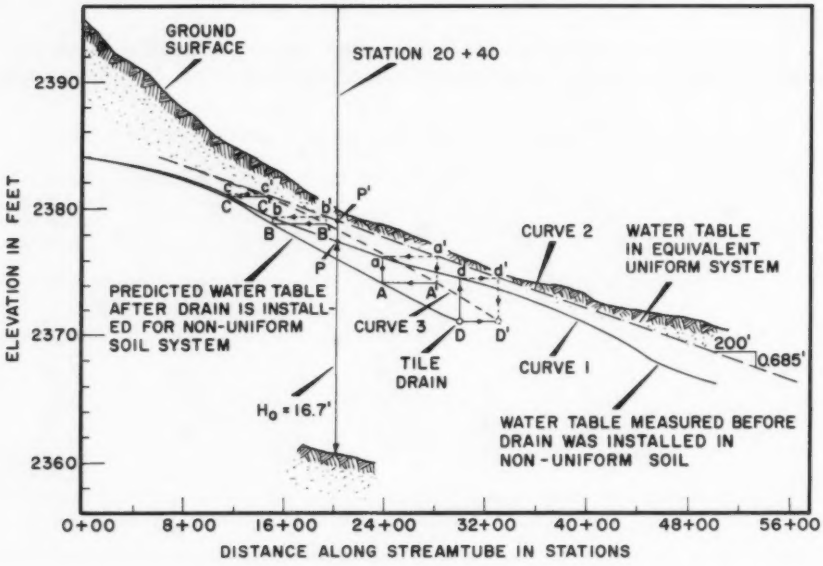

FIG. 5.—WATER TABLE PROFILE ALONG STREAMTUBE SHOWN IN FIG. 4

sects curve 1 at d. Then in accordance with the previous section, move horizontally along elevation 74.4 until the dashed line intersects Curve 2 at d'. Then downward to D' at El. 70.9 which was the original elevation for the tile line at station 30 + 00.

By this procedure the tile line has been placed in an equivalent position in the transformed uniform system. The starting point for calculating the shape of a drawdown curve for the interceptor drain has been located. The expression derived by Glover as presented by Donnan[4] and tested by Keller and Robinson,[5]

4 "Drainage of Agricultural Lands Using Interceptor Lines," by W. W. Donnan, Proceedings, ASCE, Vol. 85, No. IR1, March, 1959.

5 "Laboratory Research on Interceptor Drains," by Jack Keller and A. R. Robinson, Proceedings, ASCE, Vol. 85, No. IR3, September, 1959.

for the shape of the drawdown curve[6] was used to plot the drawdown (curve 3) in Fig. 5.

The predicted water table after drainage can be found through reverse transformation by taking individual points, for example A' on the predicted drawdown curve in the uniform medium, then moving vertically to a', the water table in the uniform case, then horizontally to a, the water table in the nonuniform system, then downward to A to the same elevation started from on curve 3. This locates point A on the water table in the actual field problem after the drain has been installed at 30 + 00. Take a second point B' on the dashed curve in the uniform media, move vertically upward to b', then horizontally to b, hence, downward to B, thereby locating another point along the water table. This procedure can be continued until enough points are located to allow a curve to be drawn representing the water table in the nonuniform media after drainage.

A possibility for inaccuracy entering the design through the reverse transformation should be noted. The saturated area available for flow is reduced by the drain drawdown. If the effective permeability of the soil in the reduced area of flow were appreciably different than that of the flow profile before drainage then inaccurate prediction would result. The two extremes of this possibility would be: (1) less pervious material overlaying very permeable material and (2) the opposite or pervious overlaying a much less permeable strata.

The natural morphological development of semi-arid soils combined with interceptor drains usually being installed at depths greater than 4-1/2 ft would suggest the first case to be the most common. In this case (item 1) the predicted water table will be above the actual case thereby simply giving a conservative design. In the less frequent case, good practice would be violated by placing the drain in the less permeable material. At greater distances from the drain this possibility for error is reduced since the drawdown is smaller. Consequently the writer would expect few cases where significant error would be introduced by this cause.

In any regular design procedure several plausible locations for a tile line should be tried in order to obtain economy in design. General experience indicates it is best to locate the drain a short distance up slope of changes from a flatter slope to a steeper slope on the water table profile (curve 1, Fig. 5). This is reasonable since the smaller the overall conductance, the steeper is the slope of the original water table.

For a complete design procedure additional streamlines in Fig. 4 should be successively considered in the manner just described. The predicted water table elevations and their respective distances along the various streamlines can be taken from the resulting profiles, such as Fig. 5, and plotted on a map of the area. By drawing contours the water table contour map of the area is obtained for the conditions expected after the drainage works are installed.

The only field data used up to this point is that required to construct a water table contour map along with some qualitative information about stratigraphy. This often is about the extent of the field data the drainage engineer has at his disposal due to the cost of conducting the drainage investigation. No direct permeability measurements using piezometer or auger hole techniques have

[6] Discussion by R. William Nelson of "Laboratory Research on Interceptor Drains," by Jack Keller and A. R. Robinson, Proceedings, ASCE, Vol. 86, No. IR2, June, 1960, p. 78.

been required, yet a water table map of the expected condition after installation of the proposed drainage works has been obtained.

DRAIN DISCHARGE

If the discharge of the proposed drain must be known as a design quantity then the additional effort and whatever complications may be associated with measuring representative soil permeability in the field is automatically required. This requirement stems from the need to know the quantity of fluid flowing in every streamtube making up the flow system (item 2 under the heading "Theoretical Basis"). Accordingly, sufficient field measurements must be made to satisfactorily establish the permeability along a water table contour line, completely traversing the region of interest. From the measured permeability values the quantity of flow in each and every streamtube making up the region can be calculated by having a measured or estimated value for the vertical dimension of the flow regime at point P. Since the width of flow is the distance between the bounding streamlines, then the conductance, K H W, is known at one section along the streamtube. For example, at station 20 + 40 along the streamtube shown in Fig. 4, by measurement with the piezometer method the permeability was found to be K_o = 0.0308 ft per hr, W_o = 90 ft as scaled from Fig. 4 and from the piezometer jettings logs H_o = 16.7 ft. The conductance K_o W_o H_o is 0.0308 (90) 16.7 giving 46.3 cu ft per hr at a unit gradient or a quanity of flow passing station 20 + 40 of 0.173 cu ft per hr. Since the permeability was measured at 20+40 (the point chosen earlier as P) then from the earlier section (Verification of Transformation) the permeability, flow depth, flow width, and water table gradient in the hypothetical uniform system are directly determined to be:

$$\left.\begin{aligned} K_o &= K_u = 0.0308 \text{ ft per hr} \\ W_o &= W_u = 90 \text{ ft} \\ H_o &= H_u = 16.7 \text{ ft} \\ M_u &= \left(\frac{d\phi}{ds}\right)_o = \frac{0.685}{200} = 0.00383 \text{ ft per ft} \end{aligned}\right\} \quad \ldots\ldots \quad (15)$$

The equation used in calculating the water table position can usually provide an estimate for the discharge of the drain in the uniform system through using the parameters of Eq. 15. The drain outflow calculated in the equivalent uniform system is equal to the flow in the heterogeneous field system. Accordingly, in the example the calculated discharge is the quantity of water which the 80 ft of drain intersecting the streamtube of Fig. 4 at station 30 + 00 will intercept. Through successive analysis of the other streamtubes in the same manner the discharge along the proposed location of a drain could be found.

CONCLUSIONS

Four things have been accomplished with respect to a design method for interceptor drains in heterogeneous soils:

1. A transformation method has been presented which allows any essentially lateral flowing system found in interceptor drainage problems to be changed into an equivalent system of uniform conducting material.
2. If the water table or characteristic flow geometry can be predicted in the equivalent uniform system, then the expected conditions after drainage can be determined through a reverse transformation.
3. The transformation and reverse transformation can be accomplished in a simple graphical way.
4. The quantity of water entering the drain is directly determined if the inflow to the drain in the uniform system can be predicted.

Accordingly, the procedures presented herein may enable more realistic design since heterogeneity of soils is rationally considered for interceptor drains. If the ever-needed test of wide field application shows the method to be beneficial and easily used, then the method will contribute to better design and greater economy in interceptor drainage.

ACKNOWLEDGMENTS

This work was conducted while the writer was employed by the Agricultural Research Service, Ft. Collins, Colorado. The encouragement of A. R. Robinson is particularly appreciated.

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Proceedings of the American Society of Civil Engineers

EFFECT OF FILTERING ON MODEL RECHARGE WELLS

By Leonard Schiff[1]

SYNOPSIS

Maintenance of injection rates is an important consideration in the use of shafts and wells for recharge or water disposal. In one experiment covering 75 days, water rose about four times higher in wells receiving unfiltered water than in wells receiving the same flow of filtered water. This was due to clogging by suspended solids carried into the wells by the unfiltered water. Filtering through 0.2 ft of filter material reduced the suspended load in the water from about 20-ppm to 1 ppm. This occurred in experiments in which the water was ponded on the filter material and in experiments in which water was made to flow over the filter material.

River water flowing over filter material passed through the filter at more than twice the rate than when ponded on the filter. Surfaces over which water flowed appeared clean for velocities of 1/2 fps. However, fines were carried into the filters as indicated by a drop in percolation rate and by losses in hydraulic head. Raking the filters to a depth of about 1/2 in. caused deposited fines to go into suspension and be carried away by the flowing water. By this simple process, good recovery occurred in the infiltration rate of filters, and thus, into the wells.

This approach envisions potential use of shafts or shallow wells with overlying filter materials strategically spaced in waterways. Maintenance would be simplified by the ability of flowing water to hold certain soil particles in

Note.—Discussion open until May 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR 4, December, 1961.

[1] Hydr. Engr., Southwest Branch, Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. of Agric., Fresno, Calif.

suspension and to pick up particles on disruption of the upper portion of the filter material.

INTRODUCTION

Shafts and wells generally must take from 1/2 sec-ft to 2 sec-ft or more to be used in ground water recharge and for water disposal. To accomplish this, high velocities of flow must and can occur in the soil surrounding shafts and wells because of the high hydraulic conductivity of the coarse soil or aquifer material selected for their installation. High velocities are also aided by the large hydraulic gradient established by the depth of water above and in the shaft or well. However, even small amounts of suspended load in the recharge water can clog aquifer material and reduce its hydraulic conductivity.

Based on observations of shaft and pit experiments, bacteriological and chemical clogging generally are less extensive and less severe than clogging due to sedimentation. This is particularly true when "good" water and aquifer materials are involved. Bacteriological clogging has been prevented by intermittent or constant chlorination and by chlorination and pumping. Bacterial activity can also be minimized by maintaining "appreciable" rather than shallow depths of water over the material through which water is infiltrating. Reduction in permeability by chemical activity has been prevented by adding amendments to the soil or water. Clogging due to sedimentation is considered in this paper.

Various methods have been used to reduce suspended loads in water prior to using a recharge or disposal facility. Suspended materials have been flocculated and allowed to settle in desilting basins. Settling, filtering, flocculating and settling, flocculating and settling and filtering are being tried or considered by such agencies as the Los Angeles Department of Water and Power, the Santa Clara Valley Water Conservation District in California, and the United Water Conservation District of Ventura County, California.[2]

The approach envisioned in this paper involves the use of relatively shallow shafts and possibly shallow wells spaced at strategic intervals in waterways. These shafts or wells may be backfilled with various materials of high porosity and covered with filter material. To further increase flow into shafts or wells, drain pipes covered with filter material could outlet into such facilities. Water could be diverted from a river or supply canal into a waterway containing shafts and wells and then the water could be returned to the river or canal. Water can be ponded temporarily, or be allowed to flow over the shafts or wells, by a regulation structure at the end of the waterway. The flowing water would not only keep the surface of the filter material clean but would maintain particles of certain size in suspension and carry away deposited particles when the surface of the filter material is disrupted.

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[2] Proceedings, Biennial Conference on Ground Water Recharge, edited by Leonard Schiff, 1959.

The experiments described herein are concerned with the effect of filtering on the performance of model recharge wells. Comparison is also made of the performance of filters when water is ponded or is made to flow over them.

PROCEDURE

Four sheet-metal filter flumes 4 in. wide, 22 in. deep, and 7 ft long, with closed ends were constructed, and are shown in Figs. 1 and 2. Figs. 1 and 2 also illustrate subsequent description of the flumes and wells. At one end of each flume is a chamber 1 ft long formed by inserting and soldering a piece of sheet metal in the flume. Holes were drilled into this piece of sheet metal and can be stoppered at various depths to regulate the position at which water can leave the chamber and enter the flume. The support holding the float-valve in the chamber can be clamped in any position to establish a given head of water in the flume. The setup permitted the entrance of water with little disturbance of the filter material. A perforated metal pipe 2 in. in diameter was inserted through a 2-in hole cut near the bottom of the end piece. This pipe extends 7 ft into the flume and touches the metal insert forming the float chamber. The perforated drain pipe was wrapped with screen. Openings in

TABLE 1.—MECHANICAL ANALYSIS OF FILTER MATERIAL AND AQUIFER MATERIAL

Material	Particle size in millimeters								
	3–5	2–3	1–2	0.5–1	0.25–0.5	0.1–0.25	0.05–0.1	0.002–0.05	0.002–>
	Percentage								
Filter	6.0	12.8	36.4	44.8	. . .	. . .	. . .	. . .	. . .
Aquifer	4.3	9.2	26.2	32.2	19.3	4.9	1.4	0.3	2.2

the screen are smaller than the smallest sand particles contained in the filter material which was placed over the drain pipe.

Metal pipes 4 in in diameter and 5 ft long were perforated and wrapped with screen. Holes were drilled in natural aquifer material in a pit at Minter Field near Bakersfield. The metal pipes were inserted into the holes to a depth of 4 ft, 7 in and served as the model wells. Each 2-in drain pipe extending from a flume outlets into a model well. The tops of the wells were capped with metal covers.

Aquifer material obtained from the pit at Minter Field was screened to eliminate all particles less than 0.5 mm. This screened aquifer material is referred to as filter material in this paper. Mechanical analysis of the filter material is given in Table 1. It was similar to that placed over aquifer material in infiltrometers previously reported as more than doubling the infiltration rate of aquifer material alone.[3] The ratio of filter material-aquifer material

[3] "The Use of Filters to Maintain High Infiltration Rates in Aquifers for Ground Water Recharge," by Leonard Schiff, extrait des Comptes Rendus et Rapports-Assemblée Générale de Toronto 1957; Internatl. Union of Geodesy and Geophysics, Gentbrugge, 1958, Vol. II, p. 207.

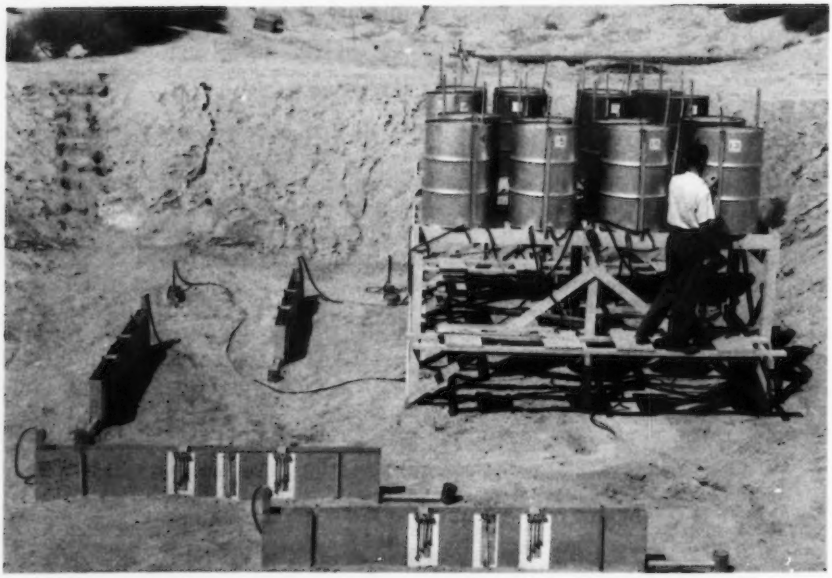

FIG. 1.—FILTER FLUMES, WITH DRAIN PIPES ENTERING MODEL WELLS

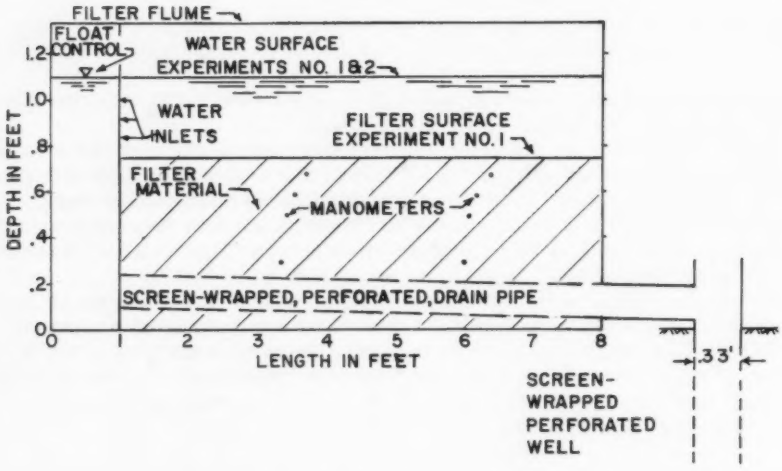

FIG. 2.—DIAGRAM OF FILTER FLUME AND WELL

FIG. 3.—WATER FLOWING OVER FILTER

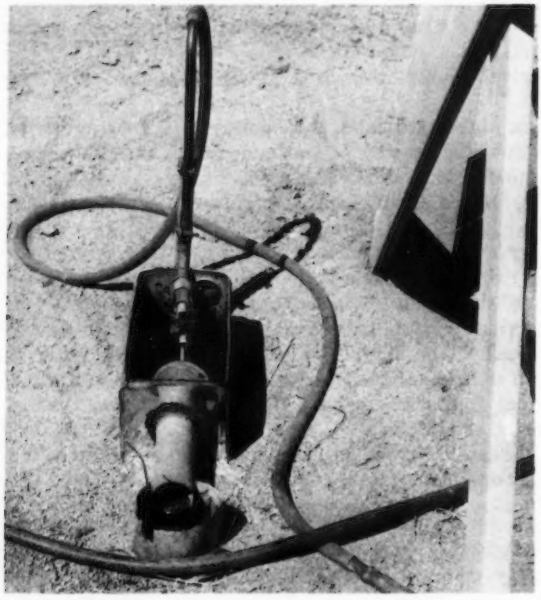

FIG. 4.—WATER INJECTED DIRECTLY INTO WELL

size appeared to be a major factor in increasing the infiltration rate along with the amount and type of suspended load and hydraulic gradient.

Manometers were connected to openings in the sides of flumes for determining hydraulic gradients to the filter material. In some flumes a slot was cut into the end piece to permit water to flow over the filter material in the flume rather than be pooled over the material as in other flumes (Fig. 3).

The rate of flow entering a well receiving filtered water was controlled by and equal to the rate passing through the filter material.

Unfiltered river water was injected directly into two wells (Fig. 4). The rates of flow into these wells were adjusted to approximate the rate of flow into wells receiving filtered water.

Once a week, quart samples of water were taken where water entered the flumes. Quart samples were also taken at the entrance to wells receiving filtered or unfiltered water. These samples were filtered in the laboratory, and the oven-dry weight of solids were determined and converted into parts per million of suspended load.

Temperature of the river water was taken whenever rates of flow into the filter material in the flumes and into the wells were determined.

SUMMARY OF RESULTS AND INTERPRETATION

Fig. 5 shows, primarily, depths of water in model recharge wells receiving filtered or unfiltered river water over a period of time. Experiment 1, listed on Fig. 5, was performed from May 14 to June 14, 1958. Experiments 2 and 3 were performed from July 7 to September 18, 1958. Suspended loads, composed of about 80% clay and 20% silt, averaged about 20 ppm for the river water entering on top of the filter material and slightly less than 1 ppm for the water entering the wells.

Unfiltered river water averaging about 20 ppm was injected directly into other wells.

The depth of water in wells is dependent largely on the volume of water entering with time, the amount of suspended load, and the nature of the aquifer material surrounding the wells. The aquifer material was essentially the same around all wells, based on mechanical analyses of samples taken at three locations.

Fig. 5 shows that in Experiment 1 filtered water entering wells occupied little depth and caused little rise in depth of water in the four wells in a 32-day period. The rate water entered wells is given in thousands of feet per day - this is the length of a circular column of water 4 in. in diameter entering a well per day; 1000 ft equal 87 cu ft or 650 gal. This is partly due to the fact that the rate of flow into wells declines with time due to clogging of the filter material.

In Experiment 2, discharge from filters was greater than in Experiment 1 and, thus, the average rate of flow into four wells (at new locations) was greater. The greater discharge was obtained by increasing the hydraulic gradient. This was accomplished by decreasing the depth of filter material over the drain pipe and increasing the depth of water on the filter material. The higher rate of flow caused a depth of about 1-ft in wells; however, this remained quite constant for the 75-day test. There was little evidence of clogging in the wells.

In Experiment 3, unfiltered river water was injected directly into two wells at the same rate that four wells received filtered water (Fig. 5). Although the

initial depth in the two wells was about the same as in the four wells, clogging caused appreciable increases in the average depth of water in the wells receiving unfiltered water, as compared to the four wells receiving filtered water.

It is important to point out that the average rate of entry of water into filter materials in all flumes dropped to about 7 ft in 32 days during Experiment 1, as compared to about 20 ft in 75 days during Experiment 2. This is partially due to differences in hydrauaic gradient, which was over twice as great in Experiment 2 as in Experiment 1.

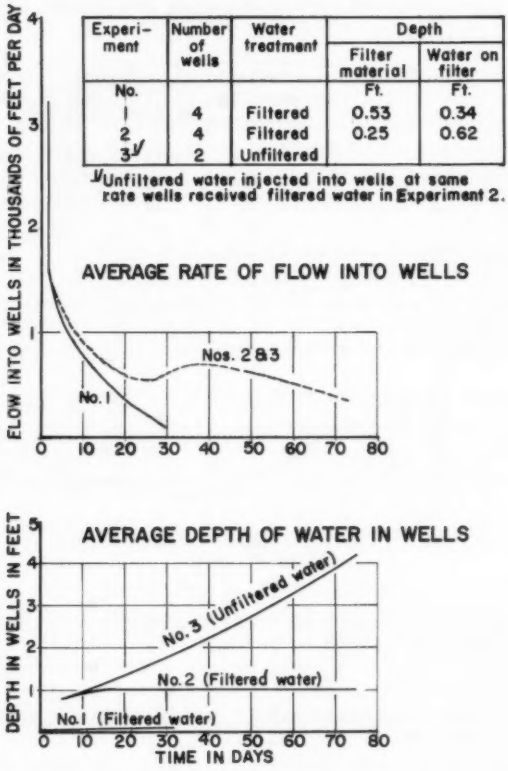
Experiment
Number of wells
Water treatment
Depth
Filter material
Water on filter
No.
Ft.
1 4 Filtered 0.53 0.34
2 4 Filtered 0.25 0.62
3 1/ 2 Unfiltered
1/ Unfiltered water injected into wells at same rate wells received filtered water in Experiment 2.
AVERAGE RATE OF FLOW INTO WELLS
FLOW INTO WELLS IN THOUSANDS OF FEET PER DAY
Nos. 2 & 3
No. 1
0 10 20 30 40 50 60 70 80
AVERAGE DEPTH OF WATER IN WELLS
DEPTH IN WELLS IN FEET
No. 3 (Unfiltered water)
No. 2 (Filtered water)
No. 1 (Filtered water)
TIME IN DAYS

FIG. 5.—DEPTHS OF WATER IN WELLS RECEIVING SIMILAR RATES OF FILTERED OR UNFILTERED WATER

Fig. 6 shows rate water enters wells after moving through filter material in flumes during the period April 25 to June 25, 1959. In two flumes water was ponded over the filter material, while in two other flumes water was made to flow over the filter material. As pointed out in the introduction, water can be pooled over shafts and wells in a waterway, or temporarily pooled and then

allowed to flow or allowed to flow continuously. In the tabular information given in Fig. 6, the depth of filter material and depth of water on filter material are the same for the tests shown. Note that in Fig. 6 the rate of discharge of filtered water from flumes (and thus into wells) was over twice as great when water was allowed to flow over filter material as when ponded over filter materials.

Velocities of flow over the "flowing" filters averaged about 1/2 fps. This velocity appeared to maintain a "clean" filter surface. The velocity was not sufficient to move any of the filter material (particles greater than 0.5 mm).

Experiments shown in Fig. 6 largely covered the months of May and June, 1959. Temperatures increased somewhat during this period, and decreases in

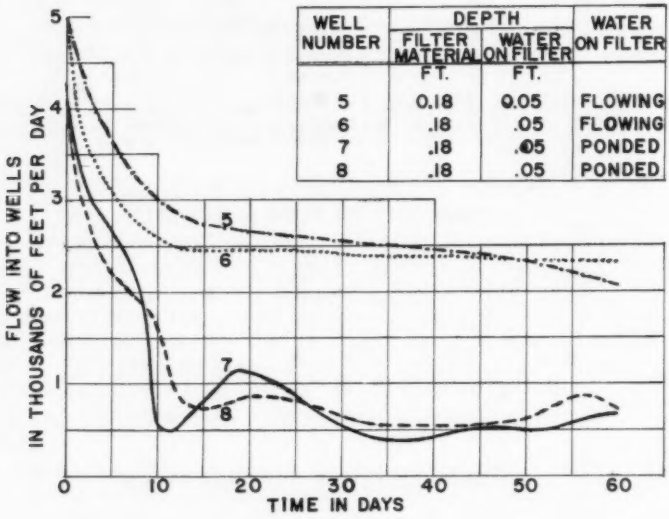

FIG. 6.—RATE WATER ENTERS WELLS AFTER MOVING THROUGH FILTER MATERIAL IN FLUMES

viscosity of the water may have tended to sustain rates at higher levels than would occur under uniform temperatures.

Maintenance of filters and operation of shafts and wells continuously for periods desired are important aspects of their use and feasibility. Although the surface of a flowing filter appeared clean, particles were carried into the filter by the infiltrating water. Clogging was evidenced by the decline in discharge rate and by hydraulic head losses, particularly near the surface of the filter material. Flowing filters lend themselves quite readily to cleaning by simply disturbing the upper portion of the filter.

Two flowing filters were run 39 days, November 1 to December 12, 1958, and 56 days, October 16 to December 22, 1958, similar to Tests 5 and 6 shown in Fig. 6. Then the filter material was disturbed by raking to a depth of about 1/2 in. with a pronged piece of sheet metal. The water flowing over the surface

of the filter became extremely cloudy with fines that had been deposited during the test. These fines were removed by the flowing water and gradually the water became clear even though raking was continued. The rates of flow through the filter material and, thus, into the well were almost completely recovered following a 39-day run and recovered from 500 to 2,400 ft per day (initially about 3,000 ft per day) following a 56-day run. In the latter case, fines may have moved somewhat deeper than 1/2 in. No increase in suspended load occurred in the water discharged from the filters during the cleaning process.

CONCLUSIONS

Where shafts or wells can be excavated through fine-to medium-textured soils to extend into coarse-textured material, a high recharge or disposal rate can result. Shafts or wells appear practical when they are backfilled and overlain with selected materials of high hydraulic conductivity. Overlying materials are selected that cause particles in the water to filter out in the upper portion of the filter material, thus preventing clogging within the shafts or wells.

Water flowing at an appreciable depth over a filter of such particle sizes as to filter out fines in the water will also tend to filter out bacteria and restrict bacteriological activity.

The filtering rate declines more slowly when water containing suspended particles flows over a filter, than when this water is ponded over a similar filter.

Disruption of the upper portion of the filter by a "raking" process after some clogging has taken place permits flowing water to remove the "clogging particles." In experiments examined herein, this method of cleaning resulted in good to almost complete recovery in filtering rate and recharge rate.

ACKNOWLEDGMENTS

The paper is a contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, United States Department of Agriculture, State of California Department of Water Resources, and California Agricultural Experiment Station Cooperating. It is also a contribution of the Task Group on Watershed Management, Committee on Water Conservation, Irrigation and Drainage Division, of the American Society of Civil Engineers.

Journal of the
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Proceedings of the American Society of Civil Engineers

DISCUSSION

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VORTEX TUBE SAND TRAP[a]

Discussion by Mashtaq Ahmad

MUSHTAQ AHMAD.[3]—The author has presented a subject of great importance to irrigation engineers especially in West Pakistan where silt exclusion is a major problem and silt excluders and ejectors have been in use for more than a quarter of a century. The writer, while making a study (15) of the silt carrying capacities of some important channels in Pakistan found that silt charge ranging from 0.2 g per litre to 10 g per litre is carried by different rivers whereas the Lacey's channel can carry only from 0.2 g per litre to 0.7 g per litre. The access silt must be excluded at the intake of the canals or through the ejectors in the canals themselves. In West Pakistan there are excluders working at Khanki and Trimmu with few others on the level crossings of the Upper Jhelum Canal. The design features are illustrated in Fig. 20. Apart from these excluders at the intake of the main canal, ejectors have been constructed in the Haveli Canal (Q=5242 cfs) and Thal Main Line (Q=6000 cfs). The designs consist of tunnels with openings facing the flow to trap bottom sediments. The actual efficiencies of these ejectors were determined and it was found that the maximum efficiency was 60% at silt intensity of 0.2 g per litre. With higher silt intensities the efficiencies dropped again. The working of these ejectors depends on the fact that the course material is carried near the bed, and if the heavily silt charged bottom flow could be withdrawn, the silt charge could be quantitatively reduced and as more of the coarser material was carried near the bottom, a qualitative reduction in silt charge would also accrue.

Before 1947, almost all the ejectors constructed in West Pakistan were of conventional tunnel type with their opening facing the flow. From 1944 to 1946 studies (16) on slit type ejectors were made in the Hydraulic Laboratories of the Irrigation Research Institute. A plain slit in the bed of a channel was placed at a distance of 10 (V_O/V_S) D downstream of hydraulic jump; where V_O and V_S are the critical and terminal velocities for the sand grade used and D is the depth of the water in the channel. In one of the series of experiments for a sand grade of 0.75 mm the discharge extraction ratio was varied from 2% to 25% for different velocities of approach in the main channel.

The conclusions were as follows:

(1) Beyond 5% to 10% extraction ratio there is little increase in the percentage silt extraction.

(2) From 2% to 5% extraction ratios the efficiency of the silt extractor begins to decrease rapidly beyond the velocities 124% to 150% above critical.

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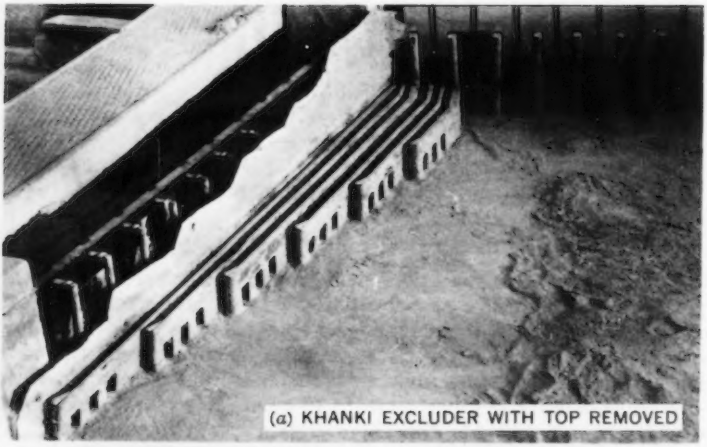
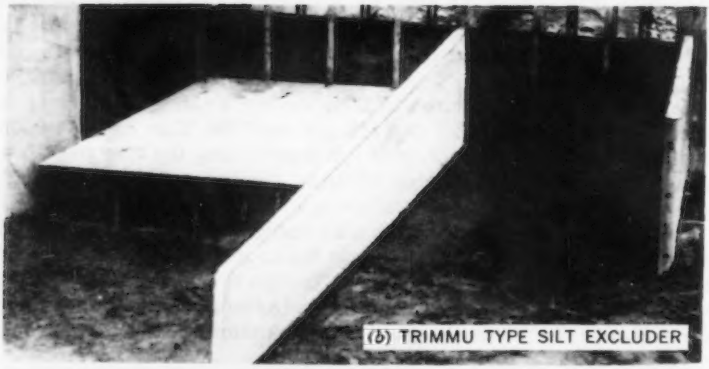
[a] December 1960, by A. R. Robinson, (Proc. Paper 2669).

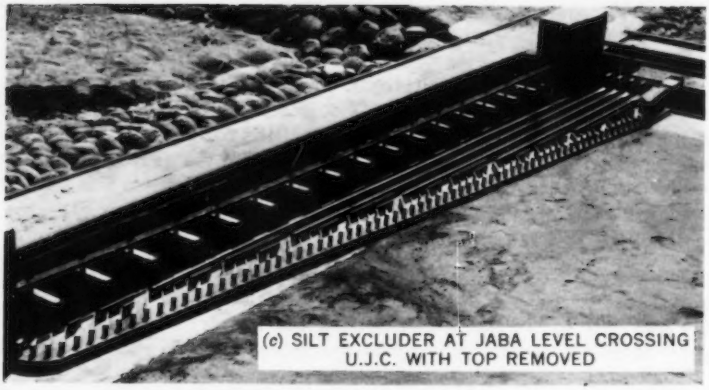
[3] Dir., Irrig. Research Inst., Lahore, Punjab, India.



(a) KHANKI EXCLUDER WITH TOP REMOVED



(b) TRIMMU TYPE SILT EXCLUDER



(c) SILT EXCLUDER AT JABA LEVEL CROSSING U.J.C. WITH TOP REMOVED

FIG. 20.—SILT EXCLUDERS AND EXTRACTORS

Slit vortex type excluders were, however, not used in this country until recently (1961). Only the conventional type of ejectors with front opening had been designed for a wide range of discharge extraction ratios varying from 2% to 40%.

It is generally important to obtain at the maximum efficiency at minimum discharge extraction ratio, because often there is little discharge to spare. A study to compare the vortex tube type silt extractor with the frontal type was started in 1951 in connection with the designs of D. G. Khan Canal ejectors. The withdrawals of vortex and conventional type ejectors as percentage of canal discharge when plotted against efficiency of silt ejector (Fig. 21), showed that a vortex type ejector can be more efficient with less discharge extraction as compared to conventional frontal type ejectors.

A basic study on design feature of vortex type ejectors had shown that the maximum efficiency of these ejectors is attained at Froude number nearly equal to 0.8 but the normal Froude number for Lacey channels were of the order of 0.2 and hence this type of ejector can only be used efficiently for West Pakistan canals if the Froude number can somehow be increased at the ejector site.

For a large canal such as D. G. Khan Canal (Q = 14720 cfs, B = 300 ft D = 12.99 cfs) experiments showed that a single slit across the canal even if placed obliquely does not work equally efficiently for its entire length. Dividing the vortex tube length into two halves equal to half the canal width at the section and extracting discharge from each half as shown in Fig. 22 gave the best results. To increase the Froude number at the slit either raise the crest or flume the structure or do both. The tests were made for D. G. Khan Canal to fix the optimum crest level and fluming ratios.

Effect of Variation of Crest Level.—In these tests on a model to a scale of 1/20, the fluming ratio was kept constant at 66%, and the radius of u/s side constriction curves at 50 ft; the crest level of the hump was raised from R. L. 431.01 and the canal bed level to 436.5 at intervals of 0.5 ft. The water surface profile was observed in each test and the diameter of the vortex tube and exit tunnels was kept equal to observed depth of water in the middle of the crest. In each test discharge equivalent to full supply discharge of canal was run for 10 hr. The water level D/S of the ejector was maintained at R. L. 444.0 and silt was injected at a uniform rate of 0.1% (by weight) by sand feeding machine. The usual observations were made of velocity distribution, surface profile, silt collections, and so forth. The afflux cause by the throated flume, which is a measure of decrease in working head across the canal head regulator, is given in Table 6. The plot of silt efficiency of silt ejector against extraction ratio of ejector for different crest R. L's (Reduced Levels) of silt ejector flume (Fig. 23) showed that the crest R. L. 435.0 was the best elevation for this structure.

The inter-relation between Froude number just upstream of vortex tube and the efficiency for different escapages of silt ejector is maximum for Froude number in the range 0.7 to 0.8 (Fig. 24). The concentrations of sediment at various points in the vertical on the axis of the flume midway between the upstream crest line and the axis of the vortex tube, were determined from samples syphoned from the flume through a pipitte of 5/16 in. outer diameter concurrently with the pitot tube velocity observations. The silt concentration in the bottom layers increases as the crest level of the silt ejector hump is raised from R. L. 431.0 to R. L. 435 or as the Froude number increases from 0.3 to 0.7.

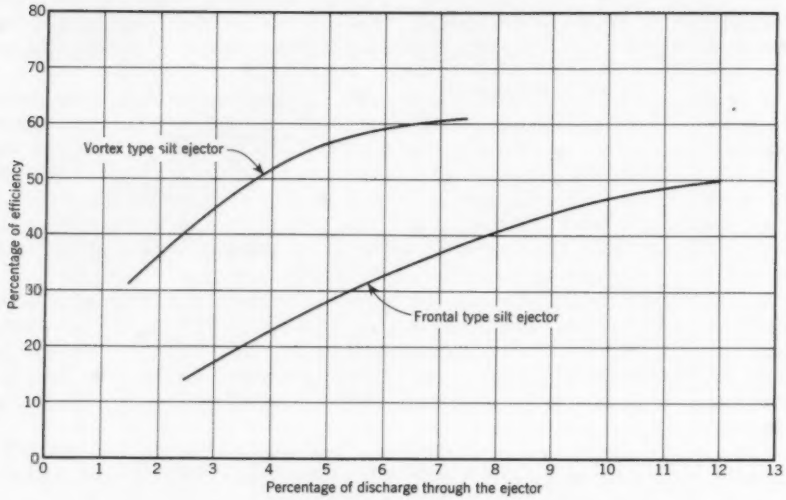


FIG. 21.—COMPARISON OF VORTEX TYPE AND CONVENTIONAL TYPE EJECTOR

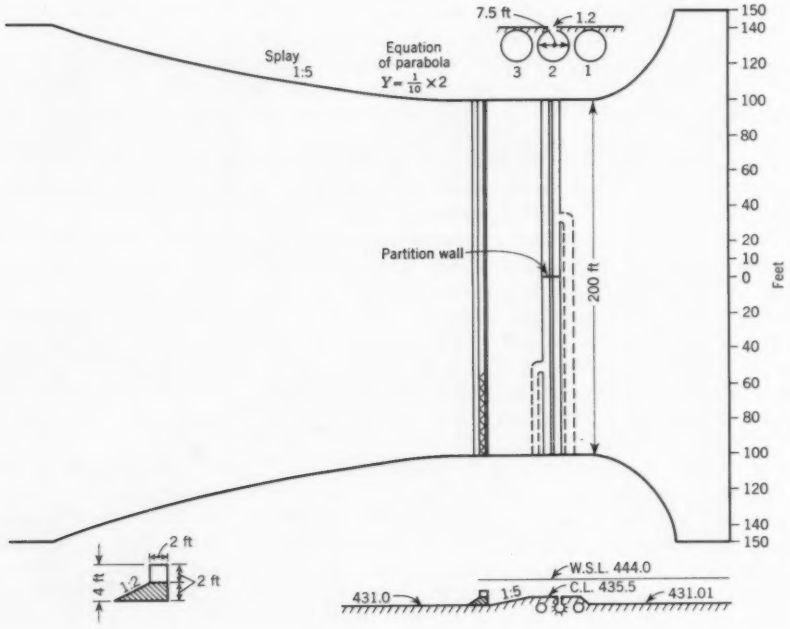


FIG. 22.—D. G. KHAN CANAL SILT EJECTOR VORTEX TUBE TYPE

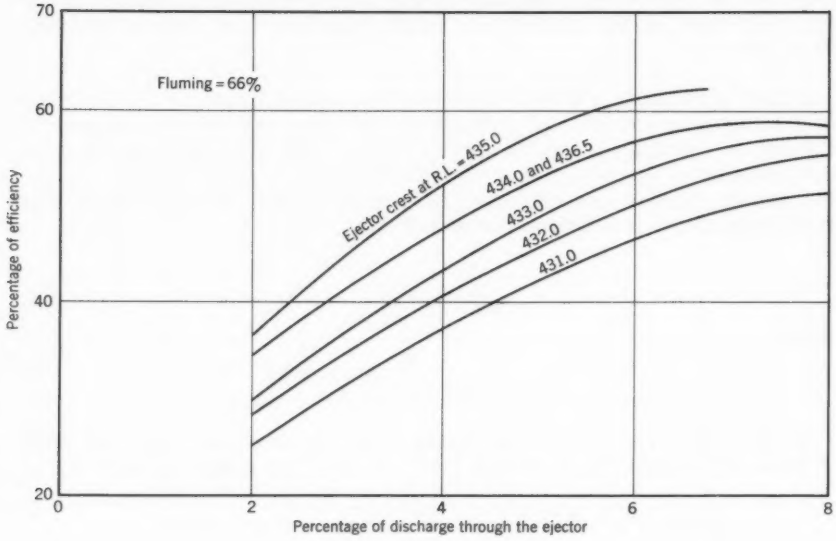


FIG. 23.—VORTEX TYPE EJECTOR D. G. KHAN CANAL EFFICIENCY AGAINST DISCHARGE EXTRACTION FOR DIFFERENT CREST LEVELS

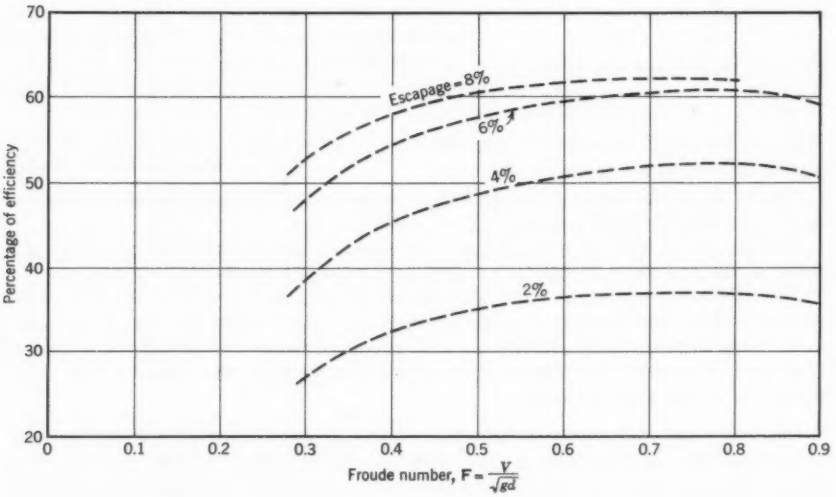


FIG. 24.—VORTEX TYPE EJECTOR D. G. KHAN CANAL EFFICIENCY AGAINST FROUDE NO. FOR DIFFERENT EXTRACTION RATIOS

It was noticed on the model that depth of flow at the crest of the silt ejector just upstream of the vortex tube nearly conforms to a velocity head given by $V_2/50$.

Effect of Fluming of Canal on the Performance of Ejector.—The efficiency of the vortex tube ejector being maximum at Froude number 0.8, the effect of fluming of the canal at the silt ejector site by 100%, 80%, 66% and 50% on the performance of the silt ejector without tampering with the optimum value of the Froude number was estimated from model tests. The crest level of the silt ejector flume was determined as follows:

$$\frac{V}{\sqrt{g\,D}} = 0.8 \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

But as

$$V = \frac{Q}{\sqrt{g}\,F\,B\,d^{3/2}} = 0.8 \quad \ldots\ldots\ldots\ldots (21)$$

As the value of Q the canal discharge, F the fluming ratio, and B the width of canal are known the value of d, depth of water just upstream of the vortex

TABLE 6.—AFFLUX CAUSED BY THE THROATED FLUME

Crest level of vortex silt ejector hump (1)	Water level u/s of silt ejector flume (2)	Afflux (3)
431.01	444.08	0.08
432.00	444.14	0.14
433.0	444.20	0.20
434.0	444.30	0.30
435.0	444.38	0.38
436.5	444.98	0.98

tube, can be computed from Eq. 21 and the value of velocity v at the throat can in turn be computed from Eq. 20.

The R. L. of the crest of the silt ejector flume = Full supply level + velocity head in approach of silt ejector = $d + (V^2/50)$.

The radius of the upstream side wing walls of the throated flume was maintained equal to $(100 - F/200)$.

The upstream radius of the hump was held equal to four times the theoretically effective depth of water at the crest of the hump. Each test for a particular fluming and escapage discharge was repeated three times and the average efficiency obtained from three tests was taken for plotting the results (Fig. 25). The conclusions are as follows:

(1) The peak discharging capacity of the ejector increases with the constriction of the canal section as the dia of the vortex tube and exit tunnels and, hence, the area of cross section of exit tunnels increases with the depth of flow over the ejector.

(2) The curves are almost parallel.

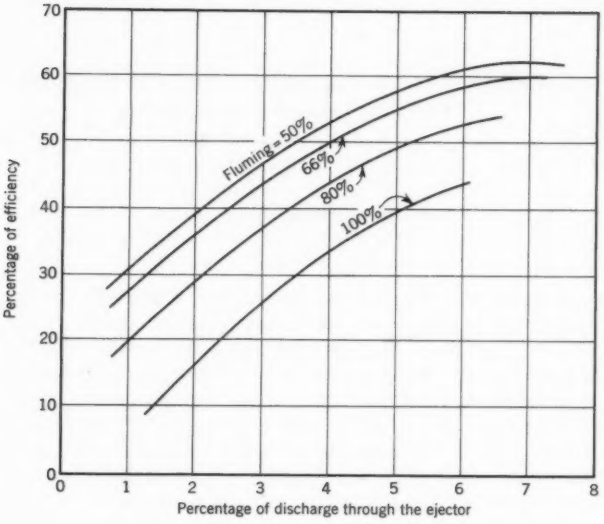


FIG. 25.—VORTEX TYPE EJECTOR EFFICIENCY AGAINST DISCHARGE EXTRACTION FOR DIFFERENT FLUMING RATIOS

(3) The efficiency of the silt ejector for an extraction ratio of 5% is 39.5%, 55% and 58% respectively, for respective fluming of 100%, 80%, 66% and 50%, showing thereby that for the same Froude number the effeciency decreases with the height of crest above canal bed level.

(4) The efficiency of the silt ejector increases with the depth of water and the downward convergence of currents, and vice-versa.

(5) The increase in efficiency by fluming beyond 66% is insignificant, which implies that no attempt should be made to increase the contraction of the canal section beyond 66% fluming.

Two vortex type ejectors have been constructed as a result of these studies, one in D. G. Khan Canal and the other in the head-race of Shadiwal Hydro-Electric Station. The latter has begun operating recently (1961) and observation of its performance and efficiency has begun.

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16. "Annual Report of Irrigation Research Institute," West Pakistan, 1946.

METHOD FOR ESTIMATING CONSUMPTIVE USE OF WATER FOR AGRICULTURE[a]

Closure by Wendell C. Munson

WENDELL C. MUNSON,[1] F. ASCE.—George H. Hargreaves, F. ASCE, points out the variation in evaporation in the southern portion of the United States even though the average monthly temperatures may be the same. This fact was considered by the writer when he noted that the assumed P. E. ratios may require adjustment to be adapted to conditions peculiar to some areas.

Harry F. Blaney, F.ASCE, questions whether the method is as simple as some of the other methods for estimating consumptive use for individual crops. This fact is acknowledged by the writer by stating in the paper that the P. E. Index method was not intended for use with individual crops but could possibly provide a rapid check of the more involved method used for this purpose.

The P. E. Index method was developed to provide a simple, rapid method for estimating Project consumptive use and still give a sufficiently accurate result to warrant its use in Project Formulation. This basic reasoning was well covered by Harold D. Hafterson, F. ASCE, who concluded his presentation with the statement, "It is wasteful to spend much time and effort to determine one portion of the problem and merely assume values for larger factors used in the solution." This statement accurately states the views of the writer.

---

[a] December 1960, by Wendell C. Munson (Proc. Paper 2672).

[1] Hydr. Engr., Bur. of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

IRRIGATION SYSTEMS OF THE TIGRIS AND EUPHRATES VALLEYS[a]

Closure by Stanley S. Butler

STANLEY S. BUTLER,[5] F. ASCE.—Vahe J. Sevian, F. ASCE, and Kamil Taj-Eddin illustrate that the newer knowledge of the United States and other advanced countries is being introduced into Iraq to such extent as is feasible under the particular economic, cultural, and political conditions of modern Iraq. They have contributed clarifying and supplemental information regarding ancient and modern irrigation developments.

The discussors state that "since 1941, floods on the Euphrates valley are sufficiently controlled by works at Habbaniyah." This calls attention to the fact that the Ramadi Barrage and the 5 ft dike around Lake Habbaniyah are merely improvements on an older Habbaniyah flood relief scheme that has been in various stages of development for decades. Previous to 1952, when the Warrar Inlet Channel was finished (Fig. 5), the flood relief was achieved by breaching the banks of the Euphrates River, allowing the flood water to flow out over the countryside toward Lake Habbaniyah. The Warrar Channel of 1952 reduced the flooding damage, and the Ramadi Barrage of 1956 increased the flood diversion discharge.

The discussors emphasize the necessity of establishing the relative rights of Iraq and the other riparian countries to the water supply of the rivers before firm plans for utilization of the remaining water in Iraq can be made. Progress along this line has been slight, but two major agreements regarding international water rights were reached among Iraq's neighboring countries during 1960—in the cases of the Nile River (Egypt versus the Sudan, November 1960), and the Indus River (India versus Pakistan, September 1960); thus, encouraging examples have been set. The main requirements seem to be patience and perseverance.

Until such international agreements are achieved, Iraq must be content with less than an ideal overall integrated plan. In the meantime the country may emphasize the development of those aspects of the plans that are relatively independent of foreign control, and, in addition, strive for increased agricultural yields through better drainage and more efficient irrigation operation and maintenance. The experience thus gained will put Iraq in a position to use the international agreements quickly whenever they may be achieved in the future. It is noted from the discussion that in Iraq water law and water rights are subject to change and growth, as in the United States, and that the trend is away from entrenched common law rights toward the rule of reasonable beneficial use of the water.

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[a] December 1960, by Stanley S. Butler (Proc. Paper 2673).

5 Assoc. Prof. of Civ. Engrg., Univ. of Southern California, Los Angeles, Calif.

INTERCEPTING DRAINAGE WELLS IN ARTESIAN AQUIFER[a]

Discussion by Mahdi S. Hantush and Closure by Dean F. Peterson, Jr.

MAHDI S. HANTUSH.[3]—The paper by Dean Peterson, F. ASCE, dealing with important problems of ground-water motion, presents analytical solutions that have considerable value in engineering application. This analytical approach is playing an increasingly significant role in the elucidation of ground-water problems, notwithstanding the objections raised by some against the idealized situations assumed in solving such problems.

In conjunction with Peterson's problems, it may be of interest to point out the following:

(a) The transient state following a sudden change in channel water level to a new permanent stage, can be shown to be given by

$$\Delta h = h(x,t) - h_x = \frac{1}{2}\left(h_n - h_o\right) f\left(\tau, \frac{x}{2A}\right)$$

$$= \frac{h_n - h_o}{2}\left\{ e^{-x/A}\operatorname{erfc}\left(\frac{\frac{x}{2A}}{\tau} - \tau\right) + e^{x/A}\operatorname{erfc}\left(\frac{\frac{x}{2A}}{\tau} + \tau\right)\right\} \quad ..(15)$$

$$\Delta q = q_t - q_o = \frac{T\left(h_n - h_o\right)}{A\sqrt{\pi}}\left\{\frac{e^{-\tau^2}}{\tau} + \sqrt{\pi}\operatorname{erf}(\tau)\right\} \quad ..... \quad (16)$$

and

$$\Delta v = \int_o^t \Delta q\, dt = S\,A\left(h_n - h_o\right)\left[\left(0.5 + \tau^2\right)\operatorname{erf}(\tau) + \frac{\tau}{\sqrt{\pi}} e^{-\tau^2}\right] \quad .. \quad (17)$$

with

$$A = \sqrt{\frac{T}{\left(\frac{K_v}{b}\right)}} \quad ..... \quad (18)$$

$$\tau = \sqrt{\frac{\nu\, t}{A^2}} \quad ..... \quad (19)$$

and

$$\nu = \frac{T}{S} \quad ..... \quad (20)$$

in which h_n represents the new permanent excess artesian head at the edge of the region; $h(x,t)$ is the excess artesian head at a distance x from the edge of

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[a] March 1961, by Dean F. Peterson, (Proc. Paper 2761).

[3] New Mexico Inst. of Mining and Tech., Socorro, N. Mex., on leave From the Coll. of Engrg., Univ. of Baghdad, Baghdad, Iraq.

the region and at time t since the start; q_t denotes the rate of channel loss at time t since the start; ΔV is the increase or decrease in the total volume of seepage occurred during a period t since the start; S represents the storage coefficient of the artesian bed; and erf, erfc equals the error function and its complement respectively.

The other symbols and the origin of the flow system are as defined in the paper.

By letting t = o in Eqs. 15 and 16, the original steady-state flow obtains (Eqs. 6 and 7b), and for large values of time ($t \to \infty$), the new steady-state flow obtains. The ultimate rate of seepage, for all practical purposes, is reached within a period given by $\tau = 2$.

Eq. 15 may be used to estimate the formation coefficients from field investigations. If the time-water level variation is observed in one or more observation holes following a change in channel water level, a type-curve method may be employed to estimate $(h_n - h_o)$, ν, and A. With these values estimated and with at least one value of Δq that is measured during the observation period, Eq. 16 affords means to solve for T and consequently for S. The type-curve method is essentially that described by Walton,[4] of the Illinois State Water Survey. In this case, however, the family of type curves are those for the function $f(\tau, x/A)$ given in Eq. 15.

(b) If, in the problem of drainage wells, the aquifer intercepted overlies a semipervious layer through which leakage is supplied from an underlying aquifer in which the artesian head remains uniform, Peterson's procedure of design may be followed provided the potential functions of Eqs. 10, 11, and 12 are replaced by their counterparts for the present flow system. The counterparts of Eq. 10 (from which the counterpart of Eqs. 11 and 12 may be obtained by the method of images), Eq. 13 and Eq. 14 are given respectively by

$$Z_1 = \frac{Q_1}{2\pi T}\left\{K_o\left(\frac{r'}{B}\right) - K_o\left(\frac{r}{B}\right)\right\} \quad \ldots\ldots\ldots\ldots \quad (21)$$

$$Z_2 = \frac{Q_2}{2\pi T}\left\{K_o\left(\frac{r_w}{B}\right) - K_o\left(\frac{r}{B}\right) + \left[1 - I_o\left(\frac{r}{B}\right)\right]\frac{K_1\left(\frac{r_e}{B}\right)}{I_1\left(\frac{r_e}{B}\right)}\right\} \quad \ldots\ldots\ldots \quad (22)$$

4 "Leaky Artesian Aquifer Conditions in Illinois," by William C. Walton, Report of Investigation 39, Ill. State Water Survey, Urbana, Ill., 1960.

and

$$Q_2 = 2\pi T \frac{\left(s_w + \frac{q_v}{\frac{K'}{b'}}\right)}{K_o \frac{r_w}{B} + \frac{K_1 r_e/B}{I_1 r_e/B}} \quad \ldots\ldots\ldots (23)$$

in which

$$B = \frac{\sqrt{T}}{\frac{B'}{b'}} \quad \ldots\ldots\ldots\ldots\ldots (24)$$

$$r' = \sqrt{(x+a)^2 + (y-b)^2} \quad \ldots\ldots\ldots\ldots (25)$$

and

$$r = \sqrt{(x-a)^2 + (y-b)^2} \quad \ldots\ldots\ldots\ldots (26)$$

and b', K' equals the thickness and the vertical hydraulic conductivity of the underlying semipervious layer; (K'/b') is the leakage coefficient (the quantity of flow that crosses a unit area of the interface between the main aquifer and its semi-confining bed under a unit excess head); K_o is the zero-order modified Bessel function of the second kind; K_1 denotes the first-order modified Bessel function of the second kind; I_o represents the zero-order modified Bessel function of the first kind; I_1 is the first-order modified Bessel function of the first kind; and s_w denotes the difference between the piezometric level in the underlying artesian aquifer and that in the pumped well.

Other symbols are as defined in the paper. In the preceding development (r_w/B) is assumed to be less than 0.01 so that $I_o\ r_w/B \approx 1$.

DEAN F. PETERSON, JR.,[5] F. ASCE.—Mahdi S. Hantush's solution for the change in piezometric level in a leaky artesian aquifer fed by a channel under a transient condition of water level, Eq. 15, should be extremely useful. Actually, water level in tidal channels continuously changes. Likewise, the treatment proposed for the well potential function in a leaky aquifer, Eqs. 21 and 22, may more accurately describe the true condition under many circumstances than the equations proposed by the writer. Hantush's ingeneous mathematical extension should materially improve the accuracy and versatility of the basic approach proposed.

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5 Prof. Civ. Engrg., Coll. of Engrg., Utah State Univ., Logan, Utah.

WATER BALANCE RECORDER[a]

Closure by H. C. Aslyng and K. J. Kristensen

HANS C. ASLYNG[11] and KNUD J. KRISTENSEN.[12]—It has been a great pleasure to read the compliments and questions by Lloyd L. Harrold, F. ASCE. Concerning the questions the writers would like to give the following remarks:

1. If the soil in the tank is not in contact with the water table, there might be more plant available water at a certain soil depth than in the adjacent field when the water table there is at a great depth. Therefore, it is not possible to record actual evapotranspiration when deep rooted crops are grown. When shallow rooted crops are grown it will probably not influence the actual evapotranspiration. Potential evapotranspiration and evaporation from bare soil will not be affected.
2. Surface run-off does not generally occur in Denmark and never when the slope, as here, is zero. The infiltration capacity is approximately 20 mm water per hr and is not exceeded by rainfall. The tank and the surrounding area may also be sprinkled without run-off and the quantity of water received by the tank is automatically recorded.

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[a] March 1961, by Hans. C. Asyling and Knud J. Kristensen (Proc. Paper 2762).

[11] Prof. and Head, Hydrotechnical Lab., The Royal Veterinary and Agric. Coll., Copenhagen, Denmark.

[12] Asst. Prof., Hydrotechnical Lab., The Royal Veterinary and Agric. Coll., Copenhagen, Denmark.

UNSTEADY FLOW OF GROUND WATER INTO DRAIN TILE[a]

Discussion by Suresh P. Brahma

SURESH P. BRAHMA.[18]—Unfortunately, Brooks has made no reference to the works of J. Boussinesq and P. Ya. Polubarinova-Kochina; this is an omission of some importance. In 1904 Boussinesq[19] solved the same problem as the author except that the drains were founded on the surface of an impervious boundary. Using the governing differential equation,

$$\frac{\partial h}{\partial t} = \frac{k}{n} \frac{\partial}{\partial x} \left(h \frac{\partial h}{\partial x} \right) \ldots\ldots\ldots\ldots\ldots \quad (14)$$

in which k is the permeability of the soil mass; n denotes porosity of the soil mass; and n represents total head, Boussinesq obtained the solution

$$h(x, t) = \frac{H f \left(\frac{x}{L}\right)}{1 + 1.115 \frac{k H t}{n L^2}} \ldots\ldots\ldots\ldots \quad (15)$$

in which H is the initial height of the water table, at a distance L from drain; $H f \left(\frac{x}{L}\right)$ is the initial shape of the free surface, and 2 L is the spacing between the drains.

Boussinesq's solution is applicable to any initial free surface configuration that can be expressed as a function of x alone.

In 1952, Polubarinova-Kochina[20] solved the identical problem considered by Brooks. In addition, she allowed for both the effects of capillarity and the size of the drains.

Finally, Brooks reported "remarkably well" agreement between his solution (which does not take into account infiltration) and A. F. Klinge's[15] field data. This is surprising, as Klinge stated that during the period of his investigations there was precipitation amounting to 2 in. to 3 in.

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[a] June 1961, by R. H. Brooks (Proc. Paper 2836).

[18] Purdue Research Foundation, Dept. of Civ. Engrg., Purdue Univ., Lafayette, Ind.

[19] "Recherches theoriques sur l' ecoutement des nappes d' eau infiltrees dans le sol," by J. Boussinesq, Journal of Pure and Applied Mathematics, Series 5, Vol. 10, 1904.

[20] "Theory of the Flow of Ground Water," by P. Ya. Polubarinova-Kochina, Gostekhizdat, Moscow, U.S.S.R., 1952.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Aerospace Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU); and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 3029 is identified as 3029(ST8) which indicates that the paper is contained in the eighth issue of the Journal of the Structural Division during 1961.

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c. Discussion of several papers, grouped by divisions.

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